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Current ACI Standards

(As of December, 1950)

Listed below are ACI standards in effect at the time the closing portions of *Proceedings* V. 46 are published in the ACI JOURNAL, Part 2 December 1950:

Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces (ACI 616-49)

ACI JOURNAL, Sept. 1949, *Proc.* V. 46, pp. 1-16. In separate covers $0.50; to ACI Members $0.40.

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)

Not adaptable to Journal presentation, this is a separate publication of large format bound to lie flat. It presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. 55 pages, $3.00; to ACI Members $1.75.

Recommended Practice for Winter Concreting Methods (ACI 604-48)

ACI JOURNAL, Sept. 1948, *Proc.* V. 45, pp. 1-20. In separate covers $0.50; to ACI Members $0.40.

Building Code Requirements for Reinforced Concrete (ACI 318-47)

ACI JOURNAL, Sept. 1947, *Proc.* V. 44, pp. 1-64. In separate covers $0.50; to ACI Members $0.40.

Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46)

ACI JOURNAL, Oct. 1946, *Proc.* V. 43, pp. 133-148. In separate covers $0.50; to ACI Members $0.40.

Recommended Practice for the Construction of Concrete Farm Silos (ACI 714-46)

ACI JOURNAL, Oct. 1946, *Proc.* V. 43, pp. 149-164. In separate covers $0.50; to ACI Members $0.40.

Recommended Practice for the Design of Concrete Mixes (ACI 613-44)

ACI JOURNAL, June 1945, *Proc.* V. 41, pp. 651-672. In separate covers $0.50; to ACI Members $0.40. Also available in a Spanish edition: "Normas Practicas Recomendadas Por el ACI Para Proyectar Mezclas de Hormigon (ACI 613-44)." In separate covers $0.50.

Specification for Cast Stone (ACI 704-44)

ACI JOURNAL, June 1945, *Proc.* V. 41, pp. 701-704. In separate prints $0.35.

Specifications for Concrete Pavements and Bases (ACI 617-44)

ACI JOURNAL, June 1945, *Proc.* V. 41, pp. 673-700. In separate covers $0.50; to ACI Members $0.40.

Recommended Practice for Measuring, Mixing and Placing Concrete (ACI 614-42)

ACI JOURNAL, June 1945, *Proc.* V. 41, pp. 625-650. In separate covers $0.50; to ACI Members $0.40.

Recommended Practice for the Use of Metal Supports for Reinforcement (ACI 319-42)

ACI JOURNAL, June 1945, *Proc.* V. 41, pp. 621-624. In separate prints $0.35.

To meet a need for authoritative information on on the use of portland cement paints as evidenced by a continuing demand for the 1942 Committee report, Committee 616 extensively revised the original report and presented a proposed recommended practice at the 1949 convention. It is now adopted as a standard without change.

ACI Standard

Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces (ACI 616-49)*

Reported by ACI Committee 616

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SYNOPSIS

This ACI standard establishes recommended practices for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage, and general characteristics and factors affecting durability.

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*Adopted as a standard of the American Concrete Institute at its 45th annual convention, February 24, 1949 as reported by Committee 616; ratified by letter ballot June 29, 1949. Title No. 46-1 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 1, Sept. 1949, *Proceedings* V. 46. Title No. 46-1 supersedes 45-18 published in January 1949. Separate prints in covers, are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Jan. 1, 1950. Address 7400 Second Boulevard, Detroit 2, Mich.

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INTRODUCTION

1. In recent years, there has been a widening utilization of paint on concrete, both as a decorative and a water repellent treatment. With it, there has developed an increased consciousness of cost, annual as well as initial. The consumer has observed that some paint jobs fail within 2 years and others last four or five times this long. He cannot afford to repaint every 2 or 3 years and wants to know what paint to use and how to get the most out of it.

2. This report has been prepared as a guide toward the realization of economy in painting concrete with portland cement paint. The effective use of oil and synthetic resin paints represents an independent problem not included within the scope of this recommended practice.

3. The term "portland cement paint" as used in this report, is understood to mean a water dilutable paint in which portland cement is the principal binder constituent. Paint that is manufactured and sold by a paint company and is ready for use except for the addition of and mixing with water is called "commercial paint". Paint that is made up by the user from the basic constituents is called "job-mixed paint". Except where specifically noted otherwise, all discussion and recommendations of this report apply to both commercial and job-mixed paints.

4. Under Recommended Procedure, there are listed the steps necessary to obtain an effective job. Appendixes 1, 2, and 3 discuss various considerations in the composition, manufacture, and durability of portland cement paint and will be valuable for a more complete understanding of what to use and how to use it.

5. The committee wishes to acknowledge especially the valuable contributions of the National Bureau of Standards and the Portland Cement Association in furnishing a large share of the basic data from which this report was prepared.

RECOMMENDED PROCEDURE

Appropriate usage

6. Portland cement paints are suitable for use on exterior and interior portland cement concrete and stucco surfaces except floors or other areas subject to mechanical abrasion. They are especially recommended where it is desired to decorate or reduce the water permeability of exterior walls built of porous, open textured concrete such as concrete block masonry. They also may be used advantageously where the concrete or stucco is damp at the time of painting or may become damp subsequently.

7. Portland cement paints are not well suited for interior surfaces requiring frequent and thorough cleaning, because coatings of these paints are not easily washed and tend to erode with vigorous scrubbing. They are of questionable value for stopping leakage through porous walls exposed to appreciable water pressure, particularly if the paint is applied to the side away from the water pressure. Better results will be obtained if the side exposed to the water is painted. Two or three heavy coats are sometimes successfully employed on the inside surface of the basement walls of houses to stop slight leakage or dampness. Severe conditions usually require more positive waterproofing methods.

8. The resistance of portland cement paints to aggressive solutions and fumes is not significantly different from that of a good quality concrete and because of their thinness they will provide only temporary protection when exposed to severe corrosive conditions. However, they are useful for protecting concrete of poor quality where the exposure is not too severe and it is feasible to repaint as often as may be required.

9. Except where the atmosphere contains reactive fumes, portland cement paint should give satisfactory service under all climatic conditions within the range experienced in the United States.

Age of concrete

10. Best results will be obtained if painting is deferred until the portland cement stucco or concrete, including concrete masonry, has aged at least three weeks. The application of tinted paints to cast-in-

place concrete walls should be postponed longer, several months after the curing period if possible, so that the concrete will be drier and a less likely source of efflorescence. Other advantages of permitting cast-in-place concrete to age longer before applying either white or tinted paints are: (a) weather tends to overcome the effects of form oil; (b) the suction of the surface is more uniform; (c) the paint film crazes less and also tends to cover any checking or crazing which may develop in the concrete surface itself prior to being painted.

Cleaning of surface

11. Surface cleaning prior to painting should consist of thoroughly removing all dirt, dust, form oil and efflorescence. Dirt and dust should be washed off with clean water. Hosing is usually adequate, but scrub brushes should be used if necessary. Soaps or other organic cleaners should be avoided, because unless completely removed they may prevent proper adhesion of the paint. To remove efflorescence first wet the concrete, then apply a 20 percent solution of hydrochloric (muriatic) acid and after about 5 minutes scour off the efflorescence with stiff bristle brushes. If necessary, a stronger solution may be used. It is best to work on small areas, not more than a few square feet at a time. The surface must be thoroughly washed with clean water after the acid treatment. It is practicable to remove traces of form oil with steel brushes, abrasive stones or lye solutions. However, if the surface is generally coated with oil, it will be more effective and economical to sandblast lightly the entire area to be painted or, if time permits, to postpone painting until the oil has been removed by weathering.

12. Some concrete surfaces are so dense (glazed) that adhesion of the paint is affected adversely. Concrete that is cast against plywood, Presd-wood or steel forms frequently presents such a problem. Before painting, such surfaces should be acid washed, lightly sandblasted, or dry rubbed with coarse grit abrasive stones, until the glaze is removed and positive "tooth" and suction for the paint are provided.

13. Old coatings of the organic type must be completely removed before applying portland cement paint. Usually this can be done most effectively and economically by sandblasting.

Wetting of surface

14. Before applying the paint the concrete must be thoroughly wetted to control surface suction and to provide a reserve of moisture to aid in the proper curing (hardening) of the paint. A garden hose adjusted to give a fine spray is well suited for this purpose. A superficial dampening with a brush dipped into a bucket of water does not supply sufficient moisture to be effective.

15. For concrete masonry walls or other concrete that readily absorbs moisture, it is usually sufficient to wet the surface in one operation not

more than one hour before painting. The spray should be applied in such manner that each part is sprayed three or four times for about 5 to 10 seconds each, depending upon the absorptivity of the surface, time being allowed between applications for the moisture to soak into the concrete; the unabsorbed water serves no useful purpose. If the surface tends to dry rapidly, as it may in hot weather, it should be redampened slightly just in advance of painting; it should be moist but not dripping wet or with a noticeable water film when the paint is applied.

16. Dense concrete absorbs moisture so slowly that it should be wetted in at least two operations not less than 30 minutes apart. It will be more effective to dampen rather large areas in advance of painting so that ample time is allowed for the moisture to soak into the concrete. When the paint is applied the concrete surface should be moist but without free water; some suction is essential and will not be provided if the concrete surface is dripping wet.

Preparation of paint

17. Portland cement paints are made ready for use by thoroughly mixing the paint powder and coarse filler, if used, with water in the proper proportions.

18. The proper amount of water will vary some with different products, largely depending upon the fineness of the dry materials. For commercial paints, the manufacturer's instructions should be followed. For job-mixed paints, the proportion of water to powder must be determined by trial. The mixed paint should have the consistency of rich cream except that a slightly thinner consistency is recommended for the first coat applied to open textured types of concrete surfaces such as concrete masonry units.

19. The proportions of water to dry material shown in Table 1 have been found to hold for a variety of paints and are suggested for trial in the absence of definite instructions:

TABLE 1

Paint, dry material, 10 lb	Water, quarts	
	For standard (rich cream) consistency	For medium thin consistency (first coat on open textured concrete)
Commercial paints (constituents mixed and finely reground)	3¼ to 4¼	3¾ to 4¾
Paints, not reground, less than 1 percent retained on No. 100 sieve	2¾ to 3¼	3¼ to 3¾
Paints, not reground, containing coarse filler; more than 25 percent retained on No. 100 sieve	2 to 2½	2½ to 3

20. Mixing procedures vary, but in general it is good practice first to reduce the dry material to a stiff paste with about half the estimated amount of water, after which additional water should be gradually stirred into the paste until the desired consistency is obtained. Preliminary reduction to a stiff paste is necessary in order to break down agglomerates of powder. When the proper proportions have been determined, it is advisable to make all batches the same except when a change in surface texture or other conditions requires a different consistency.

21. The paint should be vigorously stirred for several minutes until color and consistency are uniform and all particles are thoroughly wetted. Workability will be improved by allowing the mixture to soak or prehydrate for 30 to 45 minutes prior to application.

22. The paint should be prepared in batches of appropriate size such that it will be used up within a few hours. Most paints will remain in usable condition for about 4 hours after being prepared; however, in hot weather, some paints, especially those containing calcium chloride, should be used sooner. Keeping the mixed batch in a tightly covered container from which the painters' buckets are filled helps to prolong the usable condition.

23. While being used the paint tends to stiffen slowly due to chemical and physical reactions and evaporation of the water. Thinning the mixture with additional water to reduce consistency is perhaps not best practice, but if done with moderation, it is not considered particularly objectionable for white paints. Retempering of tinted paints should be avoided because of the tendency to alter the color. Vigorous stirring from time to time during the usable period will help to maintain proper fluidity.

Application of paint

24. No painting should be done on frozen concrete or when the paint may be exposed to temperatures below 40 F within 48 hours after application. Some paint manufacturers prefer a minimum temperature of 50 F.

25. When weather conditions are such as to cause the paint to dry rapidly, it is advisable to work "in the shade" in so far as practicable. This makes it easier to keep the surface uniformly moist for proper suction and helps prevent too rapid drying of the paint.

26. To prevent segregation and maintain a uniform mixture, the paint should be stirred frequently in the bucket.

27. The paint should be applied in two coats of the same color. Preferably not less than 24 hours should be allowed between coats; in no case should the second coat be started until the first coat has become sufficiently hard to resist marking by the brush being used. In hot,

dry weather, the first coat should be slightly moistened before applying the second coat. "Doubling back" or the application of a double thick coat is not conducive to good results and should not be permitted. Three-coat work may be employed if desired but the extra coat is seldom worth the additional cost.

28. Portland cement paints cannot be satisfactorily applied with the ordinary hair bristle paint brush. Proper application to coarse textured concrete requires a brush with relatively short, stiff fiber bristles such as fender brushes (commercially sold for cleaning the underside of automobile fenders) or ordinary scrub brushes. This is true even for certain commercial portland cement paints that exhibit smooth working qualities on the order of oil vehicle paints. If the concrete is of smooth or sandy finish, whitewash or Dutch type calcimine brushes are suitable. The paint should be brushed on in uniform thickness and free from "ridging" or excessively heavy brush marks. Laps should be well brushed out.

29. When painting concrete masonry, or other porous concrete for the dual purposes of "moisture-proofing" and decoration, both coats should be vigorously scrubbed on in such manner as to work the paint back into the voids and provide a continuous paint film free from pinholes or other openings through which water might penetrate.

30. Tests of the rain resistance of painted concrete masonry walls* indicate that paints applied with spray equipment provide less protection than scrubbed-on coatings. Spray application therefore is recommended only on dense concrete or interior surfaces or where the paint is not required for waterproofing purposes.

31. While excessively thick films are to be guarded against, there is a tendency in practice to use too much water in the paint and to brush it out too thin. Such coatings may look well at first but generally lose their opacity and protective value much earlier than thicker films. The proper average film thickness and coverage rate is difficult to estimate for portland cement paints because of the effect of differences in the texture of the concrete to be covered. However, for smooth concrete surfaces, a two-coat film should average about 0.015 in. thick; this will be obtained with a coverage rate of about 100 sq ft per gal. of mixed paint for the two coats. If the concrete is rough textured or if the paint contains coarse filler (No. 50 to 100 mesh), a film thickness of 0.025 to 0.035 in. and a coverage rate of from 45 to 55 sq ft per gal. should be sought. Two coats on very rough textured block may require as much as 1 gal. per 35 sq ft of surface. It is questionable economy to stretch the paint much further than indicated by these figures. Thinner

*Copeland, R. E. and Carlson ,C. C.;"Tests of the Resistance to Rain Penetration of Walls Built of Masonry and Concrete", ACI JOURNAL, Nov. 1939, *Proc.*, V. 36, p. 169.

films could be used, however, when repainting, provided the new and old paints are of the same color and the old paint is in fair mechanical condition.

Curing

32. Proper hardening of paint films of this type depends upon the availability of moisture for chemical reaction with the portland cement. The moisture in the concrete, in the paint itself, and in the air is utilized for this purpose but, except in very humid atmospheres, this is not enough. On most jobs it is practicable to sprinkle the painted surfaces two or three times a day with the same fog spray as used for dampening the concrete. It is recommended that this be done between coats and for at least 2 days following the final coat. The curing should be started just as soon as the paint has hardened sufficiently not to be damaged by the spray, usually about 12 hours after application. Damp curing in this or some equally effective manner must be provided in order to obtain the best results with portland cement paints. It will improve the hardness and durability of the paint in every case and in some instances will mean the difference between a satisfactory and a poor paint job.

APPENDIX 1—DISCUSSION OF COMPOSITION

1. Excluding the effect of procedure, the strength, hardness and density of the paint film are controlled primarily by the portland cement content and secondarily by the inherent durability of the other constituents and their effect on the hardening of the cement-water mixture. In addition, composition influences the appearance or decorative quality of the paint and its application properties.

2. Because of the importance of composition as a factor of paint performance, certain fundamental concepts concerning the functions and value of the various constituents commonly used in portland cement paints are included in this discussion. Available test data and service records suggest desirable percentage limits for most of the constituents, and these also have been included.

Commercial compositions

3. Based on extensive chemical analyses, commercial portland cement paints range in composition about as shown in Table 2.

TABLE 2

Ingredient	Percent by wt.		
	Min.	Max.	Typical white paint
White portland cement	20	91	61
Hydrated lime	0	60	30
Inert fillers (fine silica, limestone, etc.)	0	40	
Hygroscopic salts (calcium or sodium chloride)	0	7	3
Water repellents (stearates)	0	1.5	0.3 (or less)
Opaque pigments (zinc sulfide or titanium dioxide)	0	10	3
Water and organic matter (principally casein)	0	14	2.5 (or less)
Color pigments	0	8	0

4. One or two commercial paints contain coarse inert filler consisting of white silica sand (No. 100 to 50 mesh fraction). Such paints, by virtue of the coarser particles, produce relatively thick films having a characteristic "sandy" texture.

Requirements of Federal Specifications

5. The Federal Specification for Paint; Cement-Water, Powder, White and Tints (For Interior and Exterior Use), Designation TT-P-21* provides for two types and two classes of portland cement paint with compositional limits as indicated in Table 3 and the following paragraphs 6 and 7.

TABLE 3

Ingredient	Percent by wt.			
	Type 1 Class A		Type 2 Class A	
	Max.	Min.	Max.	Min.
Portland cement	—	65	—	80
Hydrated lime	25	—	10	—
Carbonates (calculated as carbon dioxide CO_2)	3	—	3	—
Water repellents (calcium or aluminum stearate)	1	0.5	1	0.5
Hygroscopic salts (calcium or sodium chloride)	5	3	5	3
Titanium dioxide (TiO_2) Zinc sulfide (ZnS) or mixture	5	3	5	3

6. Class B provides for 20 to 40 percent of white siliceous aggregate (No. 100 to No. 20 sieve), with the balance of the paint conforming to the requirements of Class A. Class B is designed for open-textured masonry.

7. Other provisions and requirements of the specification pertinent to this discussion are:

(a) The paint shall contain no organic binder.

(b) Color pigments shall be limeproof and shall be incorporated by a milling operation.

(c) The total free (unhydrated) calcium oxide (CaO) and magnesium oxide (MgO) in the hydrated lime shall not exceed 8 percent by weight of the hydrated lime.

(d) For dark shades (below 55 percent reflectance) the tinting material may replace the opaque white pigments, titanium dioxide and zinc sulfide.

8. It is to be noted that the typical commercial product conforms closely in composition to the Type 1, Class A paint. The specification indicates no preference between Type 1 and Type 2 except to say that the latter should be used where there is excessive moisture as in water tanks and swimming pools.

Portland cement

9. Tests and experience indicate that the most durable coatings of this type are those in which portland cement comprises a high percentage of the minus No. 100 mesh material. In the Portland Cement Association tests, straight white portland cement films showed negligible erosion and were in excellent condition after 5 years of weather exposure. Replacing 50 percent of the cement by weight with hydrated lime or fine inert filler adversely affected the hardness and weather resistance of the film. Coatings made without portland cement but with lime as the binder were softer and more rapidly eroded than those containing cement. These indications of the importance

*Obtainable from the Superintendent of Documents, Washington 25, D. C., price 5 cents.

of the cement content with regard to paint durability are in accord with experience in the field and the consensus of competent opinion expressed to the committee. Based on these considerations it is recommended that where a paint service life of 5 years or more is desired, portland cement should comprise not less than 65 percent of that portion of the paint powder that passes the No. 100 sieve. For maximum durability or for coatings to be exposed to excessive moisture as in swimming pools, it appears that this value might well be increased to 85 or 90 percent.

Hydrated lime

10. The effect of hydrated lime as a constituent of cement paint may be summarized as follows:

(a) The addition of 10 to 15 percent of lime by weight enhances slightly the whiteness and hiding power of the dry film but may reduce the opacity of the wet film. The effect is not great and depends on the quality and fineness of the lime and cement. A small amount of opaque white pigment is more effective than lime for this purpose.

(b) Workability and particle suspension of the paint mix may be improved depending on the plasticity of the lime used.

(c) In the opinion of some investigators, the use of a small amount of lime seems to increase the miscibility of some water repellents such as calcium stearate, thus shortening the time for proper mixing.

(d) Experimental data on the effect of lime content on the crazing tendency of paint films are very inconclusive except to indicate that the effect is not significant one way or the other.

(e) Depending somewhat on atmospheric conditions there is rather rapid and thorough carbonation of the lime contained in thin coatings and some fairly durable paints containing as much as 50 percent lime have been produced. However, with due consideration to such examples, it seems evident from what is now known about paints of this type that the replacement of an appreciable percentage of the portland cement with lime tends to impair the weather resistance and service life of the paint.

11. It thus appears that although lime is not an essential ingredient it probably may be safely used up to 25 percent of the portland cement content. While it cannot be said definitely that the 25 percent value should never be exceeded, the possible detrimental effects of higher lime content on the durability of exterior films seem to be more obvious than any benefits that might accrue.

Fine fillers (extenders, plasticizers, etc.)

12. The value of finely divided inert fillers such as pulverized silica, limestone, etc., is somewhat obscure. When used in appreciable percentages they have been observed to cause excessive chalking and, consequently, unsatisfactory durability and appearance with continued exposure. Being of about the same fineness as the portland cement they dilute and weaken the cement matrix. Even though such fillers may reduce shrinkage somewhat or enhance slightly such properties as brushability, it appears that their overall effect is more likely to be detrimental than beneficial.

13. Small amounts of colloidal material such as plastic clay, etc. are sometimes added to improve the workability and suspension qualities of the paint mixture. Their effect on the durability of thin cement coatings is not very well known. Probably they should be avoided or at least restricted to small amounts (2 or 3 percent) until more thoroughly tested as a paint ingredient.

14. The use of organic compounds for better dispersion and workability is not recommended except where they have been thoroughly investigated and shown not to be detrimental to the paint film.

Coarse fillers (aggregates)

15. Favorable results have been obtained with relatively coarse fillers, usually a siliceous material of acceptable color and weathering qualities such as fine white silica sand. In general the softer sands such as those derived from limestone and marble have proved less satisfactory as paint aggregate. The sand should pass the No. 20. sieve with not less than 90 percent retained on the No. 100 sieve; the No. 100 to No. 50 mesh fraction is perhaps the most satisfactory for this purpose. Because it is much coarser than the cement, such filler does not weaken the cement matrix as do the very fine fillers. The sand may be used in varying amounts but should not exceed the proportion of portland cement. Based on experiences on several jobs and on 2 to 4 year exposures of painted concrete masonry specimens included in a Portland Cement Association investigation, such filler appears to improve the surface hardness and durability of the paint. The sandy texture of the paint is considered attractive by many persons and it insures excellent adhesion between successive coats.

16. Mixtures containing sand cannot be applied with a hair brush but readily go on either smooth or rough textured concrete if a stiff bristle brush is used. The use of aggregate introduces a minor inconvenience in requiring more frequent stirring of the mixture in the paint bucket in order to keep the sand in suspension.

17. It is not recommended that aggregate be made a required constituent of portland cement paints, but its indicated value in paints that are to be applied to concrete masonry or other coarse textured concrete should not be overlooked.

Hygroscopic salts

18. Hygroscopic salts are added to portland cement paints as curing aids. Presumably they help keep the cement moist for several days with water drawn from the air, but their efficacy in this respect has not been definitely established for all conditions.

19. Calcium chloride accelerates the rate of hardening and in amounts up to 4 or 5 percent has no significant effect on ultimate strength. At this concentration, the salt apparently is not harmful as to the shrinkage, weather resistance, opacity, or other properties of the paint film. It is a possible source of efflorescence, but no evidence has been presented indicating that it is of a serious nature. Pending the development of additional data, its use in amounts not exceeding 5 percent is recommended.

20. Sodium chloride (common salt) is another salt that is sometimes used. However, it is not nearly as hygroscopic as calcium chloride and there is evidence that it tends to reduce ultimate strength. Calcium chloride is preferred also because of its definite accelerating action, which is desirable for reducing the curing period and the time during which the paint, in an unhardened state, might be damaged by rain or freezing weather.

Water repellents

21. Many of the commercial paints contain water repellents, usually either aluminum or calcium stearate powder ground with the base. Sometimes paste forms of stearate are stirred into the mixing water and added on the job. If powdered stearate is added on the job it must be very thoroughly mixed with the white portland cement and other dry ingredients prior to adding the mixing water; in such cases it is preferable and more convenient to use a waterproofed cement in which the stearate has been intimately mixed by a grinding operation.

22. The comparative merits and optimum amounts of the different water repellents as regards some properties of portland cement paint, particularly durability and shrinkage, are not well known. Some tests have been made on paints containing stearates incorporated both by grinding and by hand mixing and in addition there are some data on the effect of these substances on the strength and absorption of portland cement

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mortars. These data indicate that the amount of pure stearate in the paint should range between 0.5 to 1.0 percent of the portland cement content if water repellency and low absorption are to be obtained without too great a reduction in strength. However, some authorities contend that 0.3 to 0.4 percent of stearate is sufficient when it is ground with the paint and therefore is more homogeneously incorporated than is possible by hand mixing. Tests have shown that 2 percent of calcium stearate ground with the base seriously reduces the strength and hardness of the paint film but that 1 percent can be safely used. It also has been tentatively indicated that 1 percent of calcium stearate appreciably reduced the amount of crazing, but 0.5 percent or less was ineffective in this respect. Water repellents tend to add somewhat to the workability of the paint and the opacity of the wet film. Their effect on weather resistance has not been specifically determined as yet but it should be favorable. Proper damp curing and pre-wetting of the receiving surface are especially important when applying paints which contain water repellents because once dried they are difficult to re-wet.

23. It thus appears that small amounts of water repellents are not detrimental to the paint and offer certain advantages which may well justify their use. Not more than 1.0 percent by weight of the portland cement content is recommended.

Opaque pigments

24. White paint should contain a small amount of opaque white pigment, such as titanium dioxide or zinc sulfide, for the purpose of increasing the whiteness and hiding power of the film. Amounts of the commercially pure pigment of from 2 to 5 percent by weight of the portland cement content are sufficient and can safely be used. Due to their fineness it is possible that appreciably larger amounts might be detrimental as to shrinkage, crazing, and strength. Opaque white pigments should also be used in light tints such as cream and ivory, but are unnecessary in the darker colored paints.

Color pigments

25. Color pigments should be commercially pure oxides, very finely ground and lime proof. Different commercial grades vary in quality and price. Only the best pigments, with high tinting strength, should be used in paint. The amount of pigment required to give the color value desired depends upon its tinting strength and dispersion in the paint. For best results tinted paints should be produced in a grinding mill.

Organic substances (casein)

26. Casein or other organic binder is contained in some portland cement paints presumably to improve workability and the adhesion of the paint to nonporous surfaces. Even small amounts of such material may prevent the proper hardening of the cement and thus seriously impair the durability of the coating. Their use in portland cement paints, particularly those intended for use on exterior concrete or stucco surfaces, is therefore not recommended.

Water

27. Any clear water from a domestic source is suitable. The amount of water required to reduce the dry material to painting consistency is discussed in paragraphs 17 to 20 inclusive of "Recommended Procedure."

APPENDIX 2—MANUFACTURE AND STORAGE

1. For best results, the paint powder throughout all batches used on any one job must be uniform in composition and quality. The ingredients, therefore, must be accurately proportioned by weight, and thoroughly mixed together.

Commercial paint

2. In the manufacture of commercial paints, the mixture of finely divided ingredients generally is reground in a suitable ball mill or pebble mill. This results in better dispersion of the ingredients and thus in a more uniform product than can be obtained with ordinary job mixing methods. The finer grinding also enhances the suspension and brushing qualities of the paint.

3. Plant manufacture permits the milling in of tinting pigments with the base, which is essential for perfect uniformity of color. This is the basis for the recommendation that only commercial paints be used where colors other than white are desired. Powdered water repellents, aluminum or calcium stearate, also should preferably be milled in with the base.

4. While satisfactory results have been and are being obtained with paints prepared on the job, the convenience and advantages of using a properly-formulated commercial paint are obvious.

Job-mixed paint

5. Where a white paint is proposed, economy may sometimes be realized through the use of job-mixed paint.

6. Appendix 1 gives detailed data on ingredient proportions. It is recommended that these data be studied before determining the mix to be used. Following is a typical mix which will be found effective under normal conditions where the coating has the primary functions of decoration and water repellency:

White portland cement	75 percent
Hydrated lime	17 percent
Opaque pigment (zinc sulfide or titanium dioxide)	3 percent
Calcium chloride	4 percent
Calcium or aluminum stearate	1 percent

7. Where sand is to be incorporated with the paint, it should be held to not more than 50 percent of the total. The sand should be dry mixed with the paint powder before the gaging water is added.

8. The minimum mixing considered essential to satisfactory results consists of thoroughly stirring the dry ingredients together in a suitable container, and then sifting this mixture through a No. 14 mesh fly screen. Batches of 100 lb or more may be mixed in a small stucco or mortar mixer.

9. Paste water repellents and calcium or sodium chloride, if used, should be mixed with the gaging water.

Storage

10. Because of the hygroscopic properties of portland cement, the paint powder must be protected from the atmosphere while in storage. Particular care must be observed to maintain containers airtight where hygroscopic salts have been included in the paint.

APPENDIX 3—CHARACTERISTICS AND FACTORS AFFECTING DURABILITY

1. Portland cement paint was developed specifically for application to concrete and masonry surfaces. Since portland cement and water are the principal constituents, the resulting paint film may be considered in much the same light as any thin layer of hardened portland cement.

Resistance to moisture and alkali

2. One of the more important qualities of portland cement paint is its tolerance to alkaline conditions to which coatings on concrete are frequently subjected. Since the film and the substrate are basically the same, the alkali from the concrete that is harmful to most organic paint films has no damaging effect.

3. Prolonged saturation, alternate wetting and drying, and freezing and thawing constitute conditions harmful to any paint film, but are withstood by coatings of the portland cement type probably as well as by other types.

Appearance

4. The typical film possesses good decorative qualities as to hiding power and color. When wetted, as by rain, it becomes slightly translucent or darker in shade and appears somewhat less satisfactory than when dry. However, on drying out, the film returns to its original color and opaqueness. Tinted paint films also assume a deeper shade of color when wetted.

5. Poor opacity and color value may be due to some deficiency in the paint composition, to the use of too much water in the paint mix, to applying the paint too thin, or to the continued presence of excessive moisture behind the film.

6. The appearance of tinted paints is marred if efflorescent salts are deposited on the film surface. Much of the efflorescence trouble is caused from painting concrete at too early an age while it is still emitting appreciable excess moisture. Especially with tinted paints, it is good practice to postpone painting as long as feasible. They should not be applied to concrete that is in direct contact with water containing sulfates or other salts which might permeate to the painted surface. Frequently it is feasible to build the structure so that extraneous water will be excluded from the areas to be painted.

7. All paints suffer in appearance from normal accumulation of dirt and soot stains. Opinions differ as to whether portland cement paints are especially susceptible to soiling and staining due to their greater porosity as compared to oil and resin bound films. This matter has been under observation for some time in paint studies being conducted by the Portland Cement Association. In approximately 500 specimens exposed from 18 months to 5 years to rather bad sooting conditions the organic paints, on the average, became soiled at about the same rate as the portland cement paints.

Service life

8. By service life is meant the time required for the paint to depreciate physically to a point at which it no longer provides adequate protection or coverage. According to this concept it is not determined by a reduction in decorative value, providing that the poor appearance does not result from physical deterioration and can be remedied by cleaning rather than by repainting.

9. Based on information from several sources it appears that well-formulated portland cement paints properly applied to exterior wall surfaces exposed to the weather may be expected to have a service life of about 8 years. Some paint jobs last consider-

ably longer, some not as long. The less durable jobs frequently are those on smooth, dense concrete surfaces where there is the tendency to brush out the paint too thin and to neglect the requirements for satisfactory adhesion.

10. It is logical that paints on interior surfaces or other areas protected from the weather should last longer than when applied to exterior surfaces. Frequent cleaning however may markedly erode the film and shorten its life.

Adhesion (peeling, blistering)

11. Portland cement paint films generally adhere satisfactorily to the concrete and between coats and seldom fail by peeling or blistering. Where these defects do occur, they almost invariably are due to inadequate preparation of the surface. Portland cement paint will not adhere properly to surfaces that are glassy smooth or are soiled or coated with foreign matter. Such surfaces must be well cleaned and, if necessary, acid washed or sand blasted to open the pores and provide suction and "tooth" for the paint. A weak bond results if the concrete is not dampened prior to painting and the paint film is allowed to dry rapidly. It appears that adhesion is influenced more by differences in painting procedure than by composition although there is some evidence that flaking may be caused if the paint contains an excessive amount of unhydrated lime or water repellent.

12. Since very few cases where portland cement paints have failed by blistering are on record, little has been learned as to its cause. The presence of excess or free water on the surface being painted and the use of too much water repellent in the paint have been suggested as possible causes, but these have not been definitely verified.

Crazing

13. The typical portland cement paint film is hard, strong and relatively brittle. This latter property together with drying shrinkage sometimes results in fine crazing or checking. Exterior films seem to be more susceptible to this trouble than films which are protected from the weather.

14. Other characteristics of the film that influence crazing are tensile strength and extensibility, which in turn vary with the composition of the paint. However, the observed differences in crazing due to the ordinary variations in composition are not great.

15. Normal crazing is not progressive, generally being completed within a few weeks after starting and in itself is not a serious defect. Observations by the Portland Cement Association on several hundred test specimens over a period of 4 years showed no significant change in the extent of crazing after the first year.

16. Serious crazing, involving fractures extending through the paint film and more than 0.002 in. wide, usually is related to and induced by the development of crazing of the concrete surface subsequent to painting. Crazing over mortar joints also is usually traceable to fine shrinkage cracks in the mortar.

Chalking

17. Rapid chalking is a characteristic quality of soft, weak films and leads to proportionately rapid deterioration in the color, opacity and durability of the coating. Usually it is due either to a deficiency of portland cement in the paint or to the failure to provide adequate curing, which is necessary to the proper hydration and hardening of the cement. Soft films also may be caused by ingredients such as casein or other organic compounds, which prevent normal reaction between the cement and water.

Pinholes

18. A few pinholes in the film are not objectionable unless the paint is depended upon to reduce effectively the permeability of the surface. They result when the paint is not worked into the surface voids or when entrapped air breaks through the fresh film where it bridges instead of fills the voids. Pinholes can be eliminated by vigorous brushing. Composition is a slight factor in causing this defect only insofar as it affects the workability and surface tension of the paint. Sprayed coatings usually contain more pinholes than brush applied films, especially when the receiving surface is coarse textured or contains many deep pits or voids.

Discussion of a report by ACI Committee 616:

Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces (ACI 616-49)*

By WILLIAM M. SPURGEON and COMMITTEE

By WILLIAM M. SPURGEON†

ACI Committee 616 has produced a complete and useful standard which should do much to improve the quality of some of the portland cement paints now on the market. A few remarks may be added, based on thirty months of independent research in this laboratory, and the field experience of this company's crews in applying cement-base coatings to masonry walls.

In par. 7, p. 3, the statement is made that ". . . coatings of these paints are not easily washed and tend to erode with vigorous scrubbing." In this connection it should be pointed out that when such coatings are loaded with lime or finely divided inert fillers, such as pulverized silica, their washability is greatly impaired. Properly formulated coatings, high in portland cement content and adequately cured, offer much more resistance to washing as well as to erosion (pp. 9, 10).

In the same paragraph mention is made of the use of two or three heavy coats for dampproofing masonry walls. This could well be emphasized for the benefit of consumers, who generally do not realize that high coverage is incompatible with effectiveness as a dampproofing agent. The laws of capillary flow and of pressure flow, and field experience, show that the dampproofing effectiveness of such a coating is directly proportional to its thickness. Therefore, manufacturers of such coatings may claim *either* high coverage *or* dampproofing qualities, but they should not claim both. In no case should they claim waterproofing effectiveness (*i.e.*, against hydrostatic pressure) for brush-applied coatings.

Par. 21, p. 6, contains the statement, "Workability will be improved by allowing the mixture to soak or prehydrate for 30 to 45 minutes prior

*ACI JOURNAL, Sept. 1949, *Proc.* V. 46, p. 1. Disc. 46-1 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings* V. 46.

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to application." This characteristic is not convenient for an applicator's crews, and it is not necessary. Presumably it is recommended because calcium and aluminum stearates are not rapidly wetted by the mixing water. With modern formulation, however, water repellent coatings can be made without the use of these substances. Another possible reason for this soaking or prehydration would be the use in commercial paints of thickening agents such as starch or clays. These require a considerable period of exposure to water in order to swell and eventually become sufficiently dispersed. However, they are, in general, deleterious to cement and should not be included in cement paints. In short, we do not consider that a 30-45 minute soaking period is necessary if the paint is formulated for maximum durability.

In par. 27, p. 6, it is stated that "The paint should be applied in two coats of the same color." This is desirable, of course. If a consumer is interested in economy, however, he can get it by using gray for the first coat, as this color often can be purchased at a lower price than white or other colors.

In par. 28, p. 7, the brushability of portland cement paints is discussed. This is a subject on which further research is needed. There seems to be a dearth of information on the flow properties of portland cement suspensions in water at various concentrations and ages, with or without other substances present. The accumulation of such information would be useful for other applications of portland cement.

In par. 29, p. 7, the desirability is expressed of applying the coating in such a way as to "provide a continuous paint film free from pinholes or other openings through which water might penetrate." These openings include not only pinholes due to the method of application but also capillary passages inherent in the coating itself, resulting from its composition. Tests made in this laboratory of 22 commercial cement-base coatings mixed in accordance with the manufacturers' printed directions show 24-hour water absorption values (at age 6 days) ranging from 2 percent to 60 percent. These values are a measure of the porosities of the coating materials, the high absorption values indicating that the materials in question offer little or no resistance to penetration of water.

In par. 15, p. 11, attention is called to the desirable features of coatings made with coarse siliceous fillers. We have also found that for some coating materials the water absorption values can be considerably reduced by addition of sand. One disadvantage of sand-textured coatings should be borne in mind: their greatly increased rate of dirt collection.

The use of an accelerator such as calcium chloride, as mentioned in par. 19, p. 11, is certainly desirable. However, experience of applicators' crews indicates that the maximum value recommended, 5 percent, is too high, especially in warm weather. Calcium chloride has its greatest effect on rate of hardening of cement during the earliest stages of hydration, *i.e.*, while the cement paint is still in the mixing bucket. The result of using too much calcium chloride is that the brushable period of the paint is seriously reduced and inexperienced crews are tempted to retemper the paint. When retem-

pering is necessary, incidentally, it is best accomplished simply by washing out the brush in clean water from time to time. The brush retains enough water to thin the paint sufficiently.

The increasing use of masonry and concrete construction makes it highly desirable for architects, engineers and builders to have readily available such accurate, complete and concise information as has been supplied by this standard.

COMMITTEE'S CLOSURE

The user of portland cement paint will do well to consider Spurgeon's remarks, especially those concerned with application of the paint as a dampproofing treatment.

The need for prehydration no doubt depends to a large extent on formulation, as Spurgeon suggests. The manufacturer of the paint is in the best position to judge and his recommendations should normally be followed.

The committee is appreciative of this discussion, which constitutes a worthwhile amplification of the text of the report.

An ultrasonic method of investigating concrete deterioration and depths of surface cracks offers large possibilities in the testing of concrete structures.

An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures*

By J. R. LESLIE† and W. J. CHEESMAN‡

SYNOPSIS

A new method and apparatus for field and laboratory testing of concrete is described. The apparatus called the "Soniscope" was originally designed to detect internal cracks in concrete. It develops pulses of ultrasonic sound in the material and measures the velocity of their transmission through it. This pulse velocity has the unique advantage of being independent of the size or shape of the body under test. Measurements can be made with equal facility in mass concrete, slabs or laboratory specimens.

The existence and extent of internal cracks and the depth of visible surface cracks can be determined by the use of this apparatus.

The velocity has been found, by experiment, to be a reliable measure of the condition of the concrete and is particularly useful in deterioration studies. The dynamic modulus can be calculated from this velocity, and values so obtained are found to agree closely with the results of tests using established methods.

INTRODUCTION

The engineers of the Hydro-Electric Power Commission of Ontario, in their study of the behavior of concrete structures in service, have been faced repeatedly with the question of the seriousness and extent of the cracking that is found in most concrete structures. Some exploration of the interior of structures has been made by core drilling; the depth of visible surface cracks can be determined, if not too deep, by means of the device described by Young.§[1] These methods have been used successfully in surveying dams, but both are laborious and costly and a simpler way was sought.

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§See references p. 33.

The use of high frequency sound, or ultrasonics, was considered as offering a solution to this problem for two reasons. The transmission of high frequency sound waves in a solid body was known to be possible, using apparatus which was developed, for the most part, during the recent war. Also, it was known that when such waves are transmitted across a void in a solid body their amplitude is reduced greatly. Any abrupt change in density of the transmission path results in most of the sound waves being reflected, only a very small portion crossing the void. Therefore, during 1946 and 1947, equipment was developed which would generate the required ultrasonic vibrations and measure their amplitude after transmission through concrete.*

Since the vibrations in the concrete spread in all directions and are reflected at the boundaries, it was necessary to use pulses of sound so that the transmission of each pulse could be studied without interference from the vibrations of previous pulses.

Because the sound pulse was able to travel around voids of finite size, although with a much decreased amplitude, it was necessary to measure the time of travel in addition to the amplitude. Variations in either of these quantities were found to indicate a change in the path traveled by the pulse, and thus show the presence of a void in the body.

The depth of surface cracks was found by taking advantage of this bending of the sound path around a crack. Once the velocity in the concrete has been measured, the increased travel time required by the pulse in going around a crack can be used to calculate the path length, hence the depth of the crack.

The ultrasonic apparatus, which has been named the "Soniscope," has now been developed to the point where transmission through a 50-ft section of concrete is possible, and the velocity can be measured to an accuracy of three percent. The transmission across an air-filled crack 0.001 in. or greater in thickness has been found to be negligible; the transmission across a water-filled crack measured only 4 percent of that through a solid section of concrete.

Extensive tests have been made on four dams of the Hydro-Electric Power Commission, all of recent construction (1942-1948). These monolithic structures are characterized by lifts of 30 ft or more. With two exceptions the measurements did not reveal any internal cracks parallel to the axis of the dam, but did disclose a few deep surface cracks. Many surface cracks were examined and readings of their depth are being taken at periodic intervals.

The velocity of propagation of the sound pulse was found to differ in different specimens of concrete, there being a close correlation between pulse velocity and the modulus of elasticity of the concrete. Laboratory

*Originally described at the Research Session of the 1947 ACI Convention.

and field tests during the past two years have, without exception, confirmed this relationship. A sound theoretical basis exists for this correlation, the square of the velocity being closely proportional to the modulus of elasticity of the material (Appendix A). Simple formulas are now in use whereby the modulus can be obtained from this velocity.

The pulse method has the advantage that it does not depend on the size or shape of the specimens and can be applied equally well to mass concrete, pavement or laboratory specimens. The importance of this is evident when one considers that it permits the determination of the elastic properties of the concrete in field structures of whatever size or shape without the necessity of preparing samples therefrom. In contrast, the well-known sonic (flexural resonance) method for determining the elastic modulus has the disadvantage that the quantities recorded are dependent on the dimensions of the specimen used, which limits its use largely to prepared test specimens either laboratory made or cut from field structures. These handicaps have been partially overcome by the techniques developed by Long, Kurtz and Sandenaw[2] but their method also suffers from several limitations.

A method of obtaining an approximate value for Poisson's ratio in field structures has also been found and with this information the shear or torsion modulus is then known.

The Soniscope apparatus described herein is thus useful for obtaining three kinds of data about a concrete structure subject to the present limitation of a thickness of 50 ft: (1) the presence or freedom of internal cracks can be detected in a block of mass concrete; (2) the depth of surface cracks can be determined; and (3) the dynamic modulus of the concrete in any part of a structure can be measured in situ, regardless of the shape of the structure.

SONISCOPE APPARATUS

The equipment consists of a pulse generator containing a block of Rochelle salt crystals (Fig. 1) which is applied to one surface of the concrete and produces bursts of sound lasting a few hundred microseconds, at a frequency of about 20,000 cycles per sec, and a pulse receiver (Fig. 2) of special design which picks up the transmitted vibrations. Electronic circuits generate the pulses repetitively, about 100 times a second, and both transmitted and received pulses are displayed on a time base on the screen of a cathode ray tube, the whole assembly being mounted in a convenient carrying case (Fig. 3). The time of travel through the concrete is measured by means of a calibrated time scale on the oscilloscope screen. The rate of travel through good concrete is about 15,000 fps.

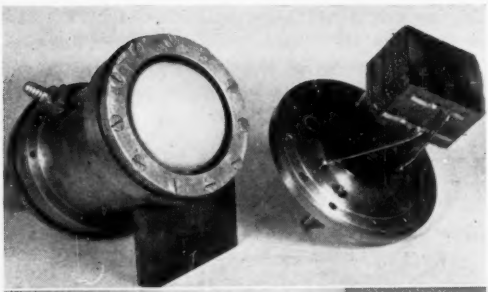

Fig. 1—Soniscope transmitter—(left) in housing, showing rubber diaphragm, (right) housing removed, showing block of Rochelle salt crystals

Fig. 2—Soniscope receiver —(left) in housing, (right) housing removed showing crystals and preamplifier

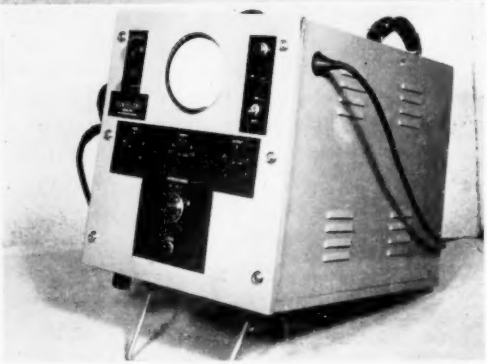

Fig. 3—Soniscope control unit

The pulse generator and receiver (Fig. 1 and 2) are connected to the control chassis by long cables. They are provided with convex rubber diaphragms which are pressed against the surface of the concrete after the latter has been wetted with oil or water.

In the case of a dam in service the transmitter is lowered down the upstream face of the dam (Fig. 4a) and the receiver is held against the downstream face (Fig. 4b).

Fig. 5a is a photograph of the oscilloscope screen showing the transmission through 52.7 ft of concrete, the outgoing pulse being at the left side of the upper trace and the received pulse at the right. Time of travel is proportional to the distance between the two and is measured to a three percent accuracy by means of a 1000 microsecond time scale on the lower trace. Any portion of the scale may be expanded by electronic means and examined in detail. In this manner, as shown in Fig. 5b, the arrival time of the received pulse was measured as 38.8 cycles or 3880 microseconds after the transmission instant, giving a velocity of 13,700 fps. A later model has a graduated dial which is calibrated directly in microseconds.

In the apparatus now in use the accuracy of velocity measurement is limited by the rounding off of the front of the received pulse (Fig. 5b) due to the attenuation of the high frequency components in the transmitted pulse. An oil path in the generator and the receiver, which is of variable length depending on the pressure the operator applies when holding the units on the concrete, also introduces an error in the time measurement.

A technical description of the apparatus is given in Appendix B.

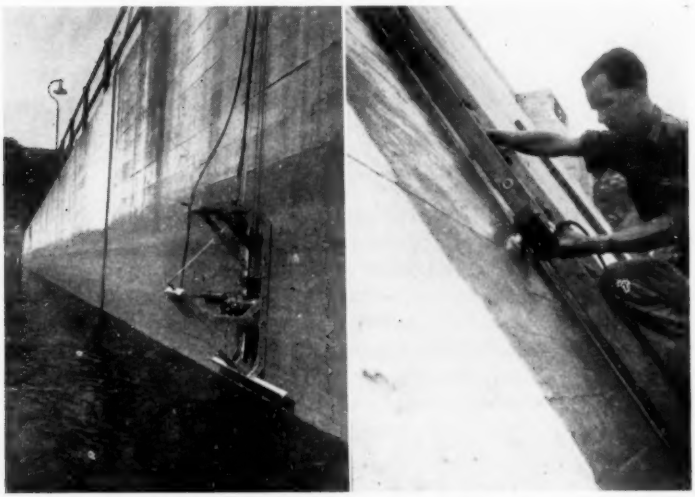

Fig. 4a (left)—Soniscope transmitter in rack on upstream face of a dam

Fig. 4b (right)—Soniscope receiver against downstream face of a dam

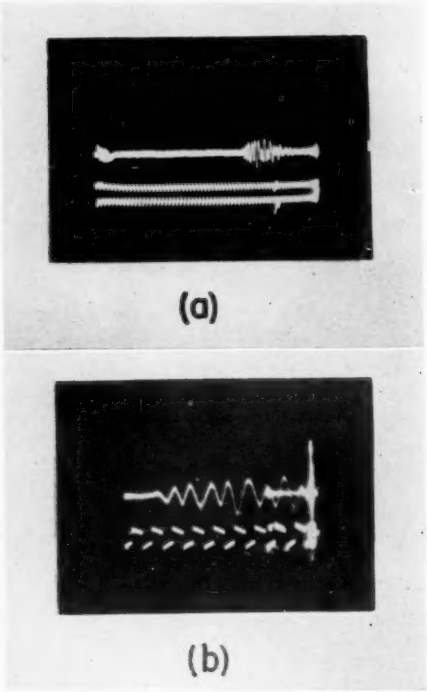

Fig. 5—Soniscope transmission through 52.7 ft of concrete:
(a) Normal sweep
(b) Expanded view of received pulse

CRACKS IN CONCRETE DAMS

Practical methods have been developed for the application of the Soniscope to the study of internal and surface cracks. It has been found that the time of transmission of the ultrasonic pulse is the fundamental measurement. In practice the amplitude of the transmitted pulses is only a secondary source of information. The reason is that the surface condition of concrete varies so greatly that the amount of sound energy transferred between the transducers and the concrete cannot be kept constant from point to point along a structure. The transmission times are, however, independent of this variation, and depend only on the path length through the concrete and the modulus of elasticity of the concrete.

In the procedure used, the pulse generator is lowered into the forebay (Fig. 4a) with the receiver held against the downstream surface of the dam (Fig. 4b) at the same elevation as the transmitter. Pulse velocities

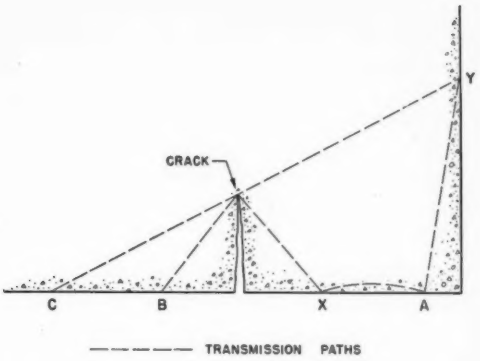

Fig. 6—Method of studying surface cracks

are then obtained through the known transmission path. In this manner the whole interior of a dam can be scanned.

If a crack or void blocks the direct path between the units, the pulse travels around the crack by diffraction, traversing a longer path in so doing, and suffering a considerable loss in amplitude. This increased travel time is then used together with the average velocity in the adjacent material to calculate the total path length. Thus the approximate extent of the crack may be found. For voids small compared to the total path length, the increased time required for travel around it may be very small. In such cases the reduction in amplitude by a factor of ten or more has been found to be sufficient to detect the void. Thus, an inspection tunnel 7 ft in cross section in a 60-ft section of a dam casts a considerable shadow at the receiver.

Variations of velocity and amplitude may, of course, be interpreted as showing the presence of a zone of low-strength concrete rather than indicating a crack. The history of the structure is therefore of great assistance in the interpretation of the velocity readings. In the case of gravity dams recently constructed under rigid concrete control conditions, it is a safe assumption that there will be no abrupt variations in modulus within one section. In such a case abnormally low velocities indicate a thermal crack.

Surface cracks

The depth of surface cracks may be obtained by placing transmitter and receiver on opposite sides of the crack. The pulse must travel down into the concrete and under the crack in order to reach the receiver. The time of travel again is used to calculate the path length. Knowing the distance between the units, the approximate depth can be obtained. This procedure has been checked in the laboratory in the case of arti-

ficially-produced cracks, and in the field by means of the drilling and dye method[1], with good agreement. The pulse method takes only a few minutes time. The accuracy in the case of deep cracks is undoubtedly low, nevertheless, the results are believed to be a fair indication of the seriousness of the crack.

The field procedure used in the case of a surface crack is illustrated with reference to Fig. 6. With the transmitter at X or Y and receiver at various positions marked A, B, C, the travel times are measured in each case.

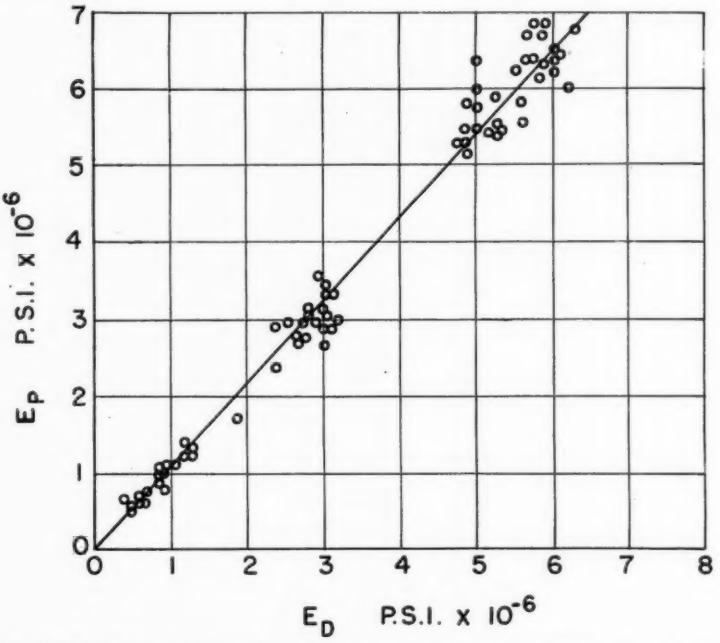

Fig. 7—Correlation between dynamic and pulse modulus for 300 laboratory beams 16 x 3½ x 4 in.

Transmission from X to A or Y to A gives the average velocity in the concrete. Travel times between X or Y and B or C are then used to calculate the crack depth.

Some 27 major surface cracks were examined in a dam under construction in 1948 and the average depth was less than 2 ft. Many other finer cracks were investigated, but found to be less than 3 in. deep. The deepest cracks were in the more recently placed sections. Cracks in

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sections over 100 days old were, on the average, only about 5 in. deep, whereas those in the newer sections had an average depth of approximately 2 ft. Periodic measurements of many of these cracks have been started to determine their behavior on a long-time basis. Measurements on a 6 year old structure show that most surface cracks have closed up to a negligible depth.

Further details regarding our experience in locating internal and surface cracks will be published when more field data is obtained. Attempts are now being made to increase the range of the equipment beyond the present 50-ft limit.

MEASUREMENT OF DETERIORATION

The velocity of compressional waves of sound bears a simple relationship to the modulus of elasticity of the material. The theoretical relations in an infinitely large mass are of the form

$$V^2 = \frac{E}{d} \frac{(1-K)}{(1+K)\ (1-2K)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

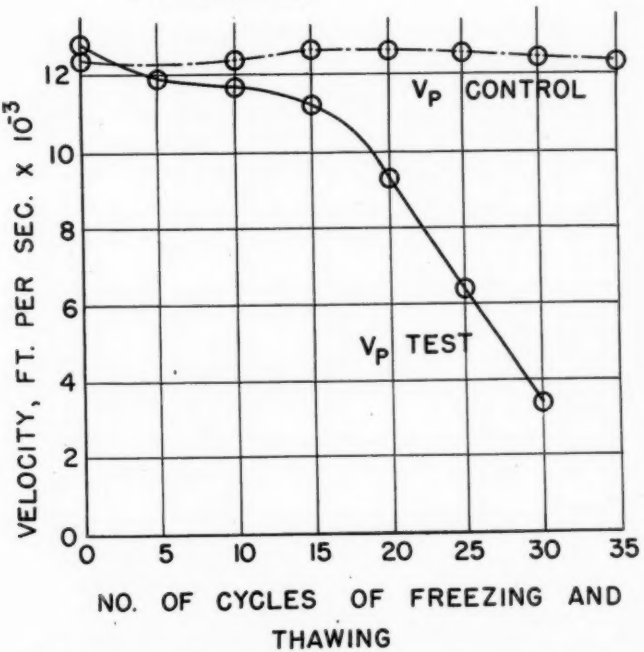

Fig. 8—Change in velocity in a specimen undergoing freezing and thawing

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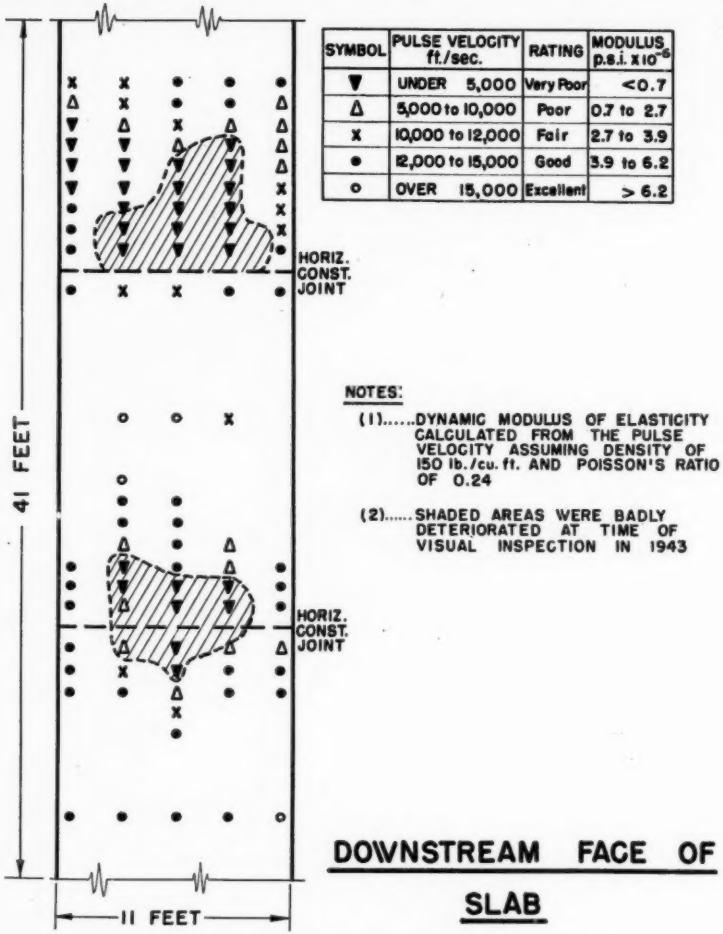

Fig. 9—Study of deterioration in one slab of an Ambursen dam

where V is the compressional pulse velocity, E is the modulus of elasticity, d is the density and K is Poisson's ratio for the material.

In the transmission of sound pulses, the wave front of the pulse has been shown to travel with this velocity, irrespective of the size or shape of the body. Thus, for concretes of average density, assuming a value for K, simple graphs can be drawn relating the pulse velocity V to E.

Extensive laboratory tests were made during 1947 and 1948 with the object of confirming this relationship. Using a series of standard 4 x 3½ x 16-in. specimens, the value of dynamic modulus calculated from the pulse velocity (using the simple formula above) was compared with the value obtained from the sonic test method (A.S.T.M. Designation C 215-47T). A high degree of correlation between the two methods was obtained (Fig. 7).

Further, it was found that for studies of durability a graph of pulse velocity against cycles of freezing and thawing gave all the information required (Fig. 8).

Proceeding on this basis, the Soniscope is now being used for the examination of concrete structures. The quantitative readings obtained can be compared with similar observations made at later times.

One of the most interesting dams tested to date with the Soniscope is an Ambursen type dam constructed in 1914. All portions of this dam have been subject to progressive deterioration, and repairs to several sections have been necessary.[3] The velocity readings were found to vary from 16,000 fps, which represents a very high modulus of elasticity, to as low as 1000 fps. A correlation of Soniscope results with those obtained by visual inspection five years previously is shown in Fig. 9 for one section. The shaded areas represent the results of the earlier inspection.

Application of the Soniscope to the study of concrete slabs has been made on pavements in cooperation with the Ontario Department of Highways. The pulse velocity was measured either from the edge of the slab to various points on the upper surface, or between two surface points. Path lengths of 3 to 10 ft were used, and where possible, measurements were made over increasing lengths of the same portion of concrete and the average velocity found.

The results of the tests made are summarized in Table 1. The pavements tested were of two groups:

1. *Test No. 1*—A recently completed dual highway in which the concrete was 2 years old. This was air-entraining concrete, made with 6 bags of cement per cu yd of concrete.

2. *Tests 2 to 13*—Older pavements varying from 16 to 24 years in age, and in conditions ranging from very bad to good. These were made of non-air-entraining concretes with cement contents of 7 or more bags per cu yd of concrete. Many of these pavements were constructed using aggregates high in chert content, resulting in very bad deterioration.

From the tabulated results, it is seen that low velocity is coincident with disintegration. In many cases of surface scaling, the underlying

TABLE 1—PULSE VELOCITY TESTS ON CONCRETE PAVEMENTS

Test No.	Compressional pulse velocity, fps	Age, years	General condition	
			Rating	Details
1	15,800 to 16,000	2	Excellent	Air-entraining concrete; 6 bags cement per cu yd of concrete; compressive strength of standard test cylinders: 7-day—2100 psi 14-day—3000 psi 28-day—3600 psi
2	16,000	18	Good	Some transverse cracking; very little scaling
3	16,200	23	Good	Some transverse cracking; very few popouts; sound aggregate
4	15,900	21	Good	Some transverse cracking; nonprogressive scaling
5	16,000	20	Fair	Slight map cracking; some disintegration and scaling at joints
6	16,200	18	Fair	Some transverse cracking
7	15,400	24	Fair to good	Some disintegration at transverse cracks; a few popouts
8	13,300 to 14,100	16	Poor to fair	Bad popouts; considerable chert in aggregate
9	8500 to 15,300	20	Poor to fair	Breaks due to chert aggregate and subgrade failure; no popouts
10	8000 to 14,000	19	Poor to fair	Disintegrating; aggregate high in chert, rotten gneiss, argillaceous limestone
11	9000 to 12,000	18	Poor	Disintegrating; aggregate high in chert and soft stone
12	7000 to 13,000	18	Bad	Many popouts; aggregate very high in chert content
13	6300 to 15,100	19	Very bad	Almost complete disintegration in places; cracking in all directions and progressive scaling; aggregate very high in chert

concrete was found to be of high strength. This scaling, which is mainly adjacent to transverse cracks and construction joints, is considered to be due to the chemicals which have been used for ice removal in the past few winters.

DYNAMIC MODULUS OF ELASTICITY

Poisson's ratio

The accurate calculation of the modulus from Eq. (1) requires a knowledge of Poisson's ratio (K). Therefore, during the laboratory comparison between the pulse and sonic methods the values of flexural and torsional moduli were used to obtain a value for Poisson's ratio (Eq. (2)). A surprising correlation between K and the density, d, was found (Fig. 10). In most calculations of E now made, the K value from Fig. 10 is used, and found to yield good results in practice.

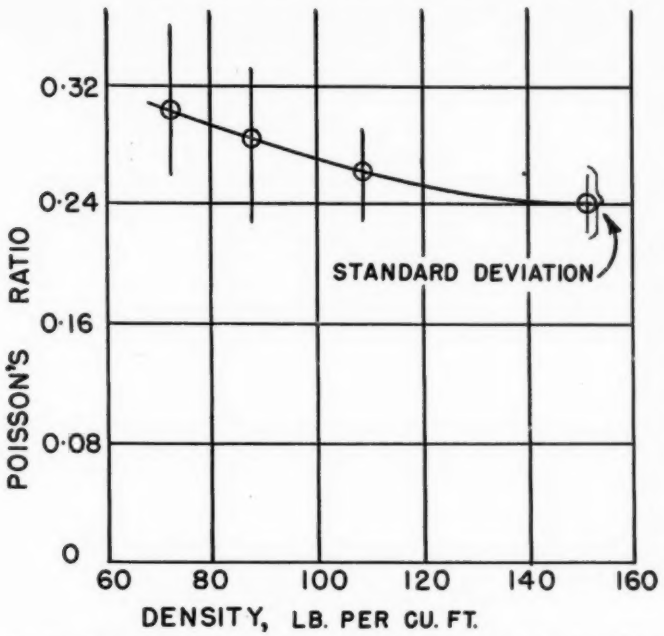

Fig. 10—Experimental relationship between Poisson's ratio and density. Water-saturated beams 16 x 3½ x 4 in.

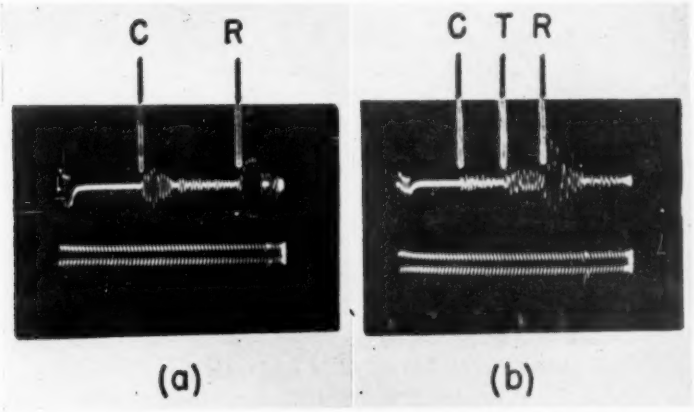

Fig. 11—Transmission of compressional, transverse and Rayleigh waves in concrete

K has also been determined in field structures. Use is made of the fact that transverse (shear) and Rayleigh waves are generated by the Soniscope generator simultaneously with the pure compressional pulse. The Rayleigh waves are sometimes called surface waves. The three types travel with different velocities and become separated in time as the path length increases. This is explained in Appendix A and is illustrated in Fig. 11 which shows the three types of waves (marked C,T,R) arriving successively at the receiver. In the example shown, the compressional and transverse waves have traveled through 30 ft of concrete and the Rayleigh wave has traveled on the surface, around the block of concrete, a distance of 50 ft. The ratio of any two of these velocities can be shown to depend only on Poisson's ratio. This relationship (Fig. 12) has been used to obtain a value for K in the field. The results were found to agree with the results of the laboratory measurements.

A value for K to be used in Eq. (1) can be taken, therefore, from Fig. 10. It should be noted that the numerical values used differ from values published previously.

Calculation of modulus

An example of the method of obtaining a value for the dynamic modulus is shown below. The measurements were made on a large gravity dam.

V_c = compressional pulse velocity = 14,370 fps
V_T = transverse pulse velocity = 8530 fps
V_R = Rayleigh pulse velocity = 8010 fps
$V_R/V_c = 0.557$
$V_T/V_c = 0.593$
d = 150 lb per cu ft (assumed)

From Fig. 12, $K = 0.212$.
Using Eq. (1) $E = 5.9 \times 10^6$ psi.

An alternative method is to use a graphical relation between V_c, V_R/V_c and E. (Fig. 13)

From Fig. 13, $E = 6.0 \times 10^6$ psi.

Had a value for Poisson's ratio been taken from Fig. 10, the results would be

$K = 0.24$
$E = 5.66 \times 10^6$

The modulus of shear or torsion may be found from the relation

$$G = \frac{E}{2(1+K)} \qquad (2)$$

FUTURE WORK

Attempts are being made to increase the accuracy of time measurement. For laboratory and small scale work, this can be achieved by a decrease in size and an increase in the resonant frequency of the units. The steepness of the transmitted wave front is increased thereby, at the expense of a decrease in range.

For large dams, an increase in transmitted power is desirable. A 60 crystal generator is being designed for this purpose. It will be 12 in. in diameter and will have directional properties.

Laboratory tests are now being carried out to determine the influence of water content and temperature on pulse velocity. The effect of reinforcing will be studied. Experience to date has, however, indicated that valid modulus readings can be made on concrete in close proximity to reinforcing.

The correlation with visual ratings of concrete in common use, and with compressive strength must be studied.

Application of the method to the testing of wood poles has been tried, and other materials, *e.g.*, rock, may be so examined.

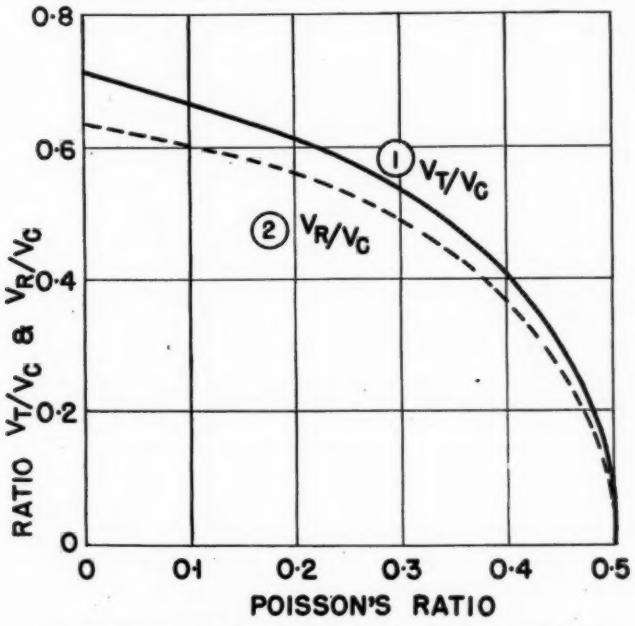

Fig. 12—Theoretical relationship between velocities and Poisson's ratio

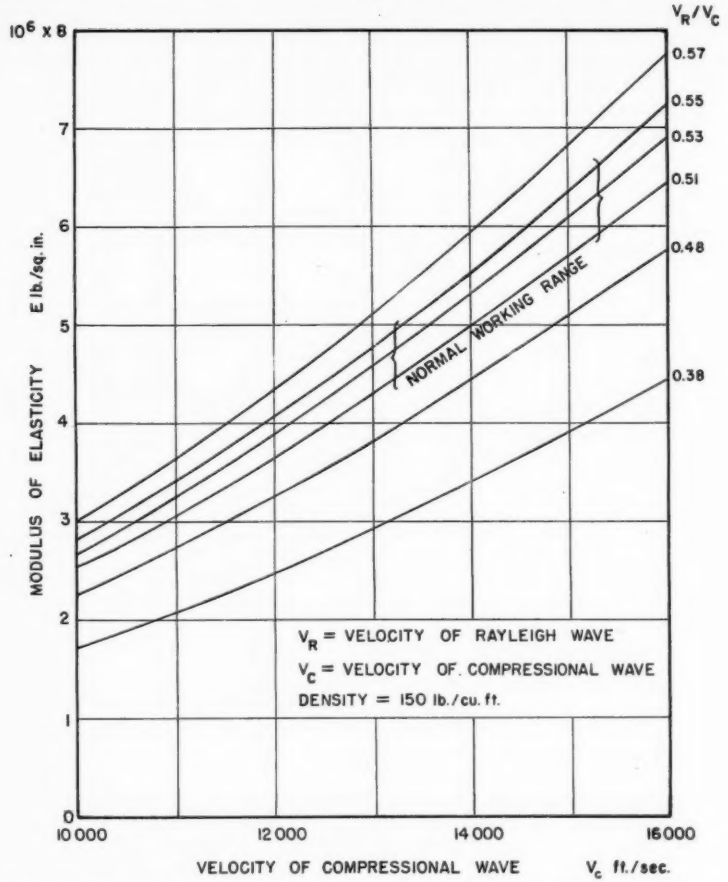

Fig. 13—Theoretical relationship between compressional velocity, surface velocity and the modulus of elasticity

CONCLUSIONS

A method and an apparatus have been described which are believed to be suitable for the detection of cracks in large concrete structures and for the determination of their approximate extent in the interior of the section. The Soniscope is also believed to be suitable for the determination of the dynamic modulus of elasticity in large field structures of concrete and in laboratory specimens. Satisfactory correlation has been obtained in laboratory tests between the so-called pulse modulus

and the normal dynamic modulus. Within the limits of accuracy of the apparatus, the pulse modulus has been found to be independent of the size or shape of the specimen under test. The method shows promise of being of great value in the study of the field performance of concrete and should prove to be an invaluable tool for long-time studies. With this apparatus the periodic measurement of the dynamic modulus of concrete in large dam sections, roads, etc., now appears possible for the first time. The modulus of odd-sized fragments of concrete can be obtained. The dynamic value for Poisson's ratio has been found to differ from the static value normally used.

ACKNOWLEDGMENTS

The authors are indebted to R. B. Young, of the Hydro-Electric Power Commission of Ontario under whose direction the work was performed, for his continued interest during the past three years; to K. H. Kidd of the Hydro-Electric Power Commission of Ontario for his contribution to the electronic circuit development; to J. V. Cole of the Hydro-Electric Power Commission of Ontario for his work on the construction of the apparatus; to E. A. Whitehurst of the Portland Cement Assn., Chicago, for his assistance in the project during the summer of 1947; and to J. Walter and J. D. Paterson of the Ontario Department of Highways for their cooperation on the tests of concrete pavements.

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APPENDIX A

Theory

Poisson, in 1828, showed that two types of waves could be propagated in an unlimited solid body, one a longitudinal or compressional disturbance, the other a transverse or shear disturbance. Their wave velocities and their relationship to the elastic constants of a material are:

$$V^2_c = \frac{E}{d} \frac{(1-K)}{(1+K)(1-2K)} \quad \text{(1a)}$$

for the compressional wave, and

$$V^2_T = \frac{E}{d} \frac{1}{2(1+K)} = \frac{G}{d} \quad \text{(2a)}$$

for the transverse (shear) wave,
where E is Young's modulus
G is the shear modulus
d is the density
K is Poisson's ratio

The boundary of an elastic disturbance will advance with either of the velocities, Eq.(1a) or Eq.(2a), and both types of waves may be present in the solid.[4] It should be noted that these velocities were calculated for an infinitely large body.

It was then shown by Lamb[5] that on the surface of a medium, a third type of wave, the Rayleigh wave, may exist. Rayleigh waves resemble water waves, their amplitude decreasing with depth. The individual particles travel in elliptical orbits, the axis of the ellipse changing with depth. They have been extensively studied in connection with earthquake waves on the surface of the earth.

Their velocity is found from the equation

$$V_R = A\ V_T \quad \text{(3a)}$$

where A is found from

$$A^6 - 8A^4 + 8(3-2Y^2)A^2 - 16(1-Y^2) = 0 \quad \text{(4a)}$$

in which $Y = V_T/V_C$ (ref. 6)

From Eq. (1a) and (2a) we find the ratio

$$\frac{V_T}{V_C} = Y = \frac{1-2K}{2(1-K)} \quad \text{(5a)}$$

Thus the ratios $\frac{V_T}{V_C} = Y$ and $\frac{V_R}{V_C}$ depend only on Poisson's ratio, K. These ratios have been plotted in Fig. 12. They are used to obtain a value for K.

The apparatus described in the paper actually measures the group velocity of sound in the body. Group velocity means the rate with which the envelope of the wave train advances through the medium. It is not necessarily the same as the wave velocity (commonly called the "velocity") in the body. As an example, in a group of deep water waves, the wave front advances with only one half the speed of the invididual waves in the group. The profile continuously changes shape as it advances and any particular wave in the group may have disappeared after a short interval.

It can be shown[7] that whenever the wave velocity V in a material varies with frequency (*i.e.*, wave length), then the group velocity, U, is not equal to V. A material in which V varies with frequency is said to be "dispersive."

The exact relation between U and V is

$$U = \frac{d(V/\lambda)}{d(1/\lambda)} = V - \lambda \cdot dV/d\lambda \quad \text{(6a)}$$

so that when $dV/d\lambda = 0$, $U = V$
(λ = wave length)

Thus if the wave velocity, V, can be shown to be independent of frequency, then the group velocity, U, is equal to V.

With the present apparatus, no change in group velocity has been observed over a frequency range in the pulse of 10 to 50 kc per sec. This supports the conclusion that the velocity is independent of frequency. Parshad[8] has reported briefly on a method of measuring ultrasonic wave velocities at much higher frequencies. The authors repeated these experiments using a 6 x 6 x 20 in. specimen. The results, although not conclusive, tended to show that the wave velocities of the three types of waves, at frequencies between 140 and 230 kc per sec were in agreement with the values used in this report. Pulse velocities and wave velocities have been assumed from the above arguments to be equal.

Several investigators have proposed, in the case of pulse measurements, that the formula relating E to V_p for laboratory specimens and slabs should be different to that used for mass concrete. The authors believe that the same formula should be used for all sizes and shapes of specimen, provided the velocities are measured using the leading edge of the pulse. Lamb[5] has shown that the leading edge of a disturbance on the surface of a solid body travels with the true group velocity, the same velocity as obtained in transmission through the body. McMillen[9] has confirmed these conclusions in an interesting experiment. The authors' measurements through and along the surface of mass concrete blocks also show that the pulse velocity is independent of the path traveled, although the accuracy of measurement requires improvement.

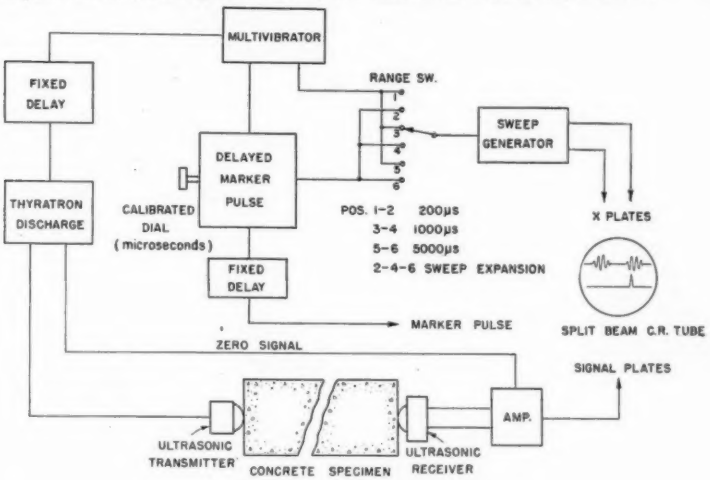

Fig. 14—Block diagram of the Soniscope

APPENDIX B

Soniscope

The pulse transmitter (Fig. 1) is an assembly of piezo-electric Rochelle-salt crystals which undergo mechanical vibration at their resonant frequency when shock excited

by a suitable voltage applied to their electrodes. The crystals are mounted in a metal housing whose transmitting face consists of a soft rubber membrane. The assembly is filled with castor oil under pressure so that the rubber diaphragm takes a convex shape. In this way the active face may be pressed against the concrete surface to provide good contact.

The crystals are shock excited by the discharge of a condenser at 800 volts through a hydrogen thyratron tube. A hermetic seal connects the crystals to a coaxial cable, and thence to the control unit.

The receiver (Fig. 2) is similar to the transmitter with the addition of a miniature battery operated amplifier mounted integrally with the housing.

The electronic circuits, shown in the block diagram, Fig. 14, provide the electrical pulses necessary for the excitation of the crystal transmitter as well as the various sweep and timing impulses.

Both transmitted and received pulses are displayed as vertical deflections on a split-beam cathode ray tube. The horizontal deflection represents time. In the later models a linear delay circuit is used to measure the time interval between transmission of the pulse and arrival of the received signal.[10] A movable marker pulse on the second trace of the oscilloscope is aligned opposite the wave front of the received pulse by means of a ten-turn helical potentiometer. Time intervals can be measured directly from the potentiometer dial to an accuracy in the latest model of one part in 1000. Any portion of the received pulse may be examined in detail by means of a sweep expansion circuit.

Discussion of a paper by J. R. Leslie and W. J. Cheesman:

An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures*

By HAROLD H. WEST and AUTHORS

By HAROLD H. WEST†

The authors are to be congratulated upon their contribution to the theory and technique of this potentially important method of testing. Inasmuch as some of the concepts introduced in their article are quite new, a discussion of these and other factors may be worthwhile, and could assist in resolving some of the problems connected with this method and suggesting avenues of research for its further development.

The fact that velocity measurements are similarly affected either by single, open cracks or by areas of low strength and general deterioration in the vibration path makes this method useful for two different types of tests, *i.e.*, crack location and measurement of general quality or deterioration. However, for a given effect upon the velocity measurement, the degree of deterioration indicated by these two conditions may differ considerably, and for deterioration measurements, at least, it is desirable to know whether the results indicate open cracks or extensive low-strength areas. As the authors pointed out, both types of deterioration may not be present in some structures, and in others the conditions responsible may be evident from visual inspection. Nevertheless, this problem may arise in connection with many structures upon which this type of testing would be most valuable, and some practical, convenient method for correlating velocity measurements with type of deterioration would greatly increase the usefulness of the method. In this connection, some investigation of the relative effects of open cracks and low-strength areas upon vibration velocity and amplitude measurements is probably desirable. Perhaps an expression involving amplitude, velocity or frequency, or some combination of these quantities, could be developed which would be useful for differentiating between these two types of deterioration.

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Data are presented by the authors showing the correlation between the modulus of elasticity computed from velocity measurements and the value obtained by the transverse-flexural (sonic) method. These data appear to verify the accuracy of the formula from which the velocity modulus was calculated. However, if the formula relating the velocity to modulus of elasticity in an infinite medium is actually correct for comparatively long, thin laboratory bars, some further explanation of this subject is required. There seems to be reasonable doubt as to whether the work of Lamb and McMillen can be used to justify the authors' choice of formulas; in the case of McMillen's experiment the test was performed with the bar immersed in a liquid medium which would give lateral restraint to the vibrations and tend to raise the velocity above that calculated by the long-bar formula. Also, in this reference the steepness of the impact wave front is not indicated, which for a specimen of fixed transverse dimensions may affect the choice of formulas. Perhaps it would be possible for the authors to provide evidence either for or against the contention that the group velocity exceeds the phase velocity in long, thin bars by comparing the wave lengths of the first and succeeding cycles.

The authors have shown that the velocity measurement provides an indication of the deterioration of the concrete tested, but it has not been satisfactorily demonstrated that the formulas relating velocity to modulus of elasticity are valid regardless of type or degree of deterioration. Consequently, it would be desirable to know whether the specimens tested and reported in Fig. 7 represented varying degrees of deterioration, and whether transverse-flexural (sonic) tests were made in conjunction with velocity tests on the freezing and thawing specimen in Fig. 8.

Values of Poisson's ratio used by the authors vary from 0.24 to 0.31, which is high compared with the value of 0.166, normally used for the transverse-flexural (sonic) method. Seemingly, the same value of Poisson's ratio should be used for both velocity and flexural methods, and this difference should, therefore, be resolved, or if separate determinations are required on individual specimens, this fact should be established.

In using the velocity method for locating cracks, it should be pointed out that at frequencies below those used by the authors, a greater proportion of the vibrations can be transmitted across water-filled gaps. At frequencies of about 1000 cycles it may be impossible to detect the presence of a narrow, water-filled crack in the vibration path and this must be given consideration in designing an instrument to perform its desired function.

Despite the limitations outlined by the authors and in preceding paragraphs, it is felt that the velocity method of dynamic testing can be a useful one for the investigation of concrete in place, and that with the acquisition of more knowledge and experience in this field, present limitations on use and interpretation of the test can be removed or greatly extended.

AUTHORS' CLOSURE

West has examined the paper in great detail and has recognized the difficulties which have faced the authors in their investigations. Many of the questions which he brings up have not been answered with finality at the present time.

It is true that in theory the operator has an option in interpreting a low velocity reading obtained in a particular investigation. He may attribute the reading to the presence of either an open crack or a zone of low strength material. In the structures examined to date, the authors have not experienced difficulty in this regard. The type of cracking which has been the principal subject of these investigations (thermal shrinkage cracks) is that which appears in mass concrete shortly after construction, so that the cracks are surrounded by sound, high velocity material. Deterioration is revealed as low velocities over an area, irrespective of the transmission paths used. In practice it is found that the vibration amplitude depends to a marked degree on the surface at the points of application of the transducers, hence it can be used only as a secondary indication of the quality of the concrete.

West questions the author's choice of formula in applying the instrument to long, thin specimens. The criterion used to date is that reflections from the boundaries do not arrive at the receiver in time to interfere with the directly transmitted wave front. The steepness of the wave front is, therefore, the critical quantity. Laboratory specimens 16 x 3½ x 4 in. can be measured satisfactorily with a pulse rise time of about seven microseconds, corresponding to a natural frequency in the transmitter of 35,000 cycles per second. The cycles following the first received half-cycle are distorted by the interference of those waves which are affected by the boundaries. Experimentally we have not found the arrival time of the initial disturbance to be affected by the distance to a boundary.

Regarding the accuracy of the formula in the case of badly deteriorated

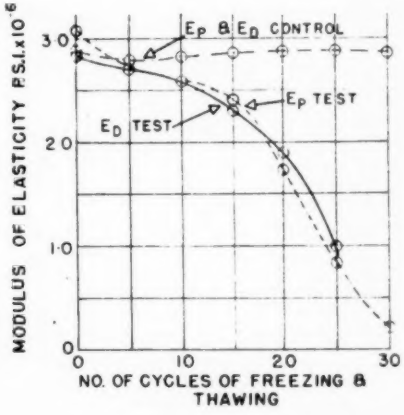

Fig. A—Comparison between "pulse modulus" and dynamic modulus during deterioration of lightweight concrete

concrete, the results quoted in Fig. 7 include lightweight concrete broken down by laboratory freezing and thawing. The comparison between the "pulse modulus," E_P, and the usual dynamic modulus, E_D, determined by the sonic (flexural) method, for one of these lightweight specimens, during deterioration, is shown in Fig. A. (This is the same specimen as shown in Fig. 8.)

The values of Poisson's ratio quoted in Fig. 10 were obtained from sonic tests using the formula $K = \frac{E_D}{2G} - 1$. Three lightweight and one standard concrete were in the series, which used 16 x 3½ x 4 in. specimens, soaked in water at room temperature for 24 hours before testing. It should be noted that these values of K agree with those obtained in mass concrete from the ratios V_R/V_C and V_T/V_C.

It is agreed that the choice of frequency determines the thickness of flaw which can be detected. Our choice is a compromise between the highest possible frequency, to detect cracks, and the lowest possible, to get adequate transmission range in concrete.

The authors believe that a wide field of application for pulse techniques exists in materials testing and research. Exploratory tests have been made on materials such as soils, rocks, wood and asphalts.

Carpenter's glue sustained air bubbles during initial set of plastic foam concrete.

Manufacture of Reinforced Foam Concrete Roof Slabs*

By I. T. KOUDRIASHOFF†

SYNOPSIS

The Russian type of lightweight concrete described used a rosin-glue emulsion to preserve the air voids before the initial set of the cement. Shrinkage was decreased and strength increased through high pressure steam curing. The autoclave treated foam concrete used in the production of precast industrial roof slabs had a unit weight of 47 lb per cu ft and a compressive strength of over 500 psi. The lightweight slabs, used in a load carrying capacity and as insulation, reduced construction time by 50 percent and costs by as much as 20 percent. Test data on roof slabs and production procedures are also described.

INTRODUCTION

Foam concrete has been used in the USSR as insulating material for walls and roofs since 1930. The first concrete of this type was made with cement, water and a rosin-glue emulsion consisting of glue and rosin-soap. The cement required amounted to 350-380 kg per cu m (6.3 to 6.8 sacks per cu yd) and the dry emulsion substances 160-200 g per cu m (0.27 to 0.34 lb per cu yd). The unit weight varied from 400 to 450 kg per cu m (25 to 28 lb per cu ft), compressive strength from 4 to 6 kg per sq cm (57 to 85 psi) as measured in 7 cm (about 2¾ in.) cubes. The coefficient of thermal conductivity varied from 0.07 to 0.09 cal per sq m per m per hr per deg C (0.56 to 0.72 BTU per sq ft per hr per deg F per in.). The foam concrete was placed over a supporting surface either as a monolithic slab or as smaller slabs, obtained by sawing 1.0 x 0.5 x 0.25 m (40 x 20 x 10 in. approx.) precast blocks to the desired size and shape.

The production and use of foam concrete in this manner was found to be uneconomical because of the large quantity of cement required. Also

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the material developed high shrinkage stresses producing cracks; had a low compressive strength; and was easily broken during handling.

Consequently, in 1936 the Central Institute for Industrial Building Research, Moscow, in collaboration with the designing board "Promstroiproject" studied the problem of using reinforced foam concrete as supporting slabs as well as an insulating material for industrial building roofs.

According to specifications, industrial building roofs shall consist of reinforced foam concrete slabs from 10 to 12 cm (4 to 4.8 in.) thick, placed on metal girders. Over the slabs a leveling layer of cement mortar from ½ to 1 cm (0.2 to 0.4 in.) thick shall be placed, followed by a mat of roofing felt saturated with bitumen. The weight of foam concrete shall be between 600 and 700 kg per cu m (38 and 44 lb per cu ft) and its compressive strength shall be not less than 20 kg per sq cm (285 psi).

As a result of the research carried out in the Concrete Laboratory of the Central Institute for Industrial Building Research during the period between 1936 and 1938, an autoclave treated reinforced foam concrete for load carrying members was developed, with improved physical and engineering properties: cement requirements varied from 195 to 300 kg per cu m (3.5 to 5.4 sacks per cu yd); unit weight of concrete from 550 to 850 kg per cu m (34.4 to 53.0 lb per cu ft); compressive strength of 10 cm (3.94 in.) cubes from 20 to 52 kg per sq cm (284 to 740 psi) and thermal conductivity from 0.08 to 0.24 cal per sq m per hr per deg C per m (0.65 to 1.94 BTU per sq ft per hr per deg F per in.).

PREPARATION AND MIXING

The production of a cellular, fine porous, foam-concrete mass is essentially a mechanical mixing operation of the cement mortar with foam. The foam must be firm and stable so that it resists the pressure of the mortar until the cement takes its initial set and a strong skeleton of concrete is built up around the voids filled with air.

The development of reinforced foam concrete was based upon two new factors: (1) the addition of ground sand to cement, which reduced the required amount of cement by ⅓ to ½, and (2) high pressure steam treatment of foam concrete in autoclaves, greatly increasing the strength of the concrete and reducing the effects of shrinkage.

Foam

Rosin soap, made by mixing sodium hydrate and rosin in a 1:6 ratio is one of the most effective foam generators. To increase the strength and stability of the walls of foam bubbles, the rosin soap was mixed with carpenter's glue, using a rosin-glue ratio of 1:1½. Foam generators using soap root, alginate, naphtha soap and lupine albumen are also

known, but the rosin-glue emulsion using cheap, commercial materials was more widely used in the production of industrial roof slabs.

The rosin-glue emulsion was made by preparing the carpenter's glue solution, boiling the rosin soap and then mixing the glue solution with the rosin soap.

Glue solution was prepared by soaking the glue in water for 1 to 1½ hours at 40-50 C (104-122 F) until a homogeneous liquid mass was obtained.

Rosin soap was prepared as follows: the required quantity of sodium hydrate was diluted with water to a density of 20 degrees (according to Baumé) as checked by an areometer. Then the alkali solution was brought to the boiling point and the rosin, previously broken into small pieces not larger than 5 mm (0.2 in.), was gradually poured into the boiling solution which was continuously stirred during pouring. The mixture was boiled for 1½-2 hours until the rosin was completely dissolved and the rosin soap had a homogeneous color without lumps and grains.

After cooling the rosin soap to 60 C (140 F), the glue solution, heated to 40 C (104 F), was poured in in small portions, and continuously stirred. The resulting emulsion was stored in a closed wooden container.

Cement mortar

The sand used for the cement mortar was a quartz sand, either ground or unground (rock flour or dust-like quartz), 95 percent of which passed the 900 mesh per sq cm (US No. 70) sieve and 60 percent the 4900 mesh per sq cm (US No. 170) sieve.

A local siliceous sand called marshalite (rock flour) was also investigated and found to be satisfactory as an aggregate. The marshalite and a fine sand found in large deposits nearby were used unground after being passed through a 64 mesh per sq cm (US No. 20) sieve. To aid in the prevention of settling, the maximum size of ground sand was 0.6 mm (0.024 in.).

The cement-sand ratio could vary from 1:1 to 1:3 depending upon the quality of the cement, the fineness and the chemical composition of the sand and the desired strength and unit weight of the concrete.

Mixing

Special machines used in mixing the foam concrete consisted of three drums: (1) an emulsor for whipping the rosin-glue emulsion into foam, (2) a mortar mixer for preparing the cement paste and (3) an agitator to mix the foam with the paste to produce the foam concrete.

MANUFACTURE OF UNITS

Plant

The plant set up for the fabrication of roof slabs was divided into the following sections:

Storage—Three bins were provided for cement, sand and marshalite, a steam dryer for drying the sand and marshalite, and 2 sieves with 64 mesh per sq cm (US No. 20) for grading materials.

Fabrication—The equipment included a table for cleaning and preparing forms and placing the reinforcement, a roller conveyor for moving the forms after placement of reinforcing steel to a three drum mixer with a 250 liter (8.8 cu ft) capacity, scales for proportioning materials, and small gage rail cars to transport the poured slabs to the curing department.

Curing—The area used for curing the slabs before steam treatment was equipped with railway tracks for storing the cars loaded with slabs.

Steam treatment—Three autoclaves 7 to 8 m (23 to 26 ft) long and 1.8 m (5.9 ft) in diameter and a control room equipped with a steam distributor for regulating the supply of steam from the boiler room to the autoclaves and the dryer constituted the steam treating department.

Fabrication of slabs

In developing the work of the new plant, the foam concrete mixture was proportioned as follows: cement, marshalite, sand and water in ratios varying from 1:1:1:1 to 1:1:2:1.2. The cement content varied from 155 to 300 kg per cu m (2.8 to 5.4 sacks per cu yd) of foam concrete.

Several types of metal forms were tested under working conditions and forms made of roofing iron were adopted because of their lightweight,

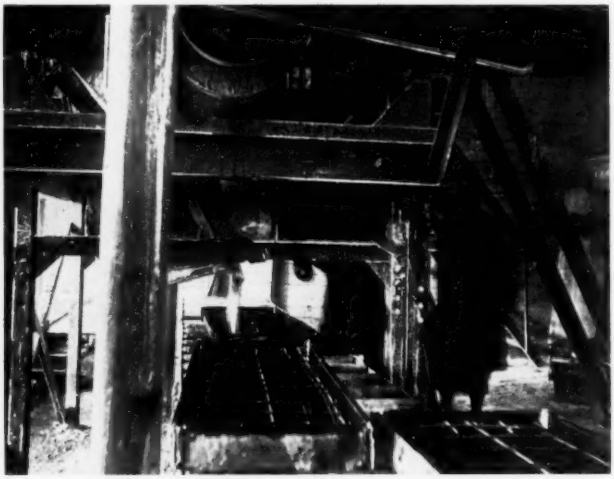

Fig. 1—After placing the reinforcement, the forms were filled with foam concrete.

Fig. 2—The filled forms were placed on small gage rail cars and then leveled with wooden floats.

ease of removal and low cost. Forms were lubricated with oleic acid in two parts of kerosene.

Reinforcement consisted of smooth iron bars 6 mm (¼ in.) in diameter, 10-12 cm (4 to 5 in.) on centers, top and bottom of the slab, with transverse rectangular loops of 3 mm (⅛ in.) bars spaced 15 cm (6 in.) on centers, tying the top and bottom bars into one solid frame. The weight of reinforcement was 56 kg per cu m (94 lb per cu yd). Protective concrete on the bottom reinforcement was 2 cm (0.8 in.) thick and on the top reinforcement 1 cm (0.4 in.) thick. At the supports, the reinforcement extended almost to the ends of the slab to provide reinforcement at points of bearing.

Operating procedure—Metal forms were cleaned, lubricated and tied at the corners by means of an asbestos packed clamp. The reinforcing framework was placed in the form which was moved on the roller conveyor under the hopper of the foam mixer (Fig. 1). The foam concrete was placed in the form which was then carried by two men to a car (Fig. 2) standing on railway tracks near the mixer. The top surface of the slab was then leveled with a wooden float. Cars were loaded with 26 slabs, which was all the head room of the autoclave would allow. To increase the stability of the loaded car and to allow free passage of steam between forms, layers were separated with 3 or 4 flat iron hooks 40 mm (1.6 in.) wide and 15 mm (0.6 in.) thick (Fig. 3).

After a car was loaded, it was moved to the curing area before steam treatment. Slabs had to be cured for at least 12 hours before being

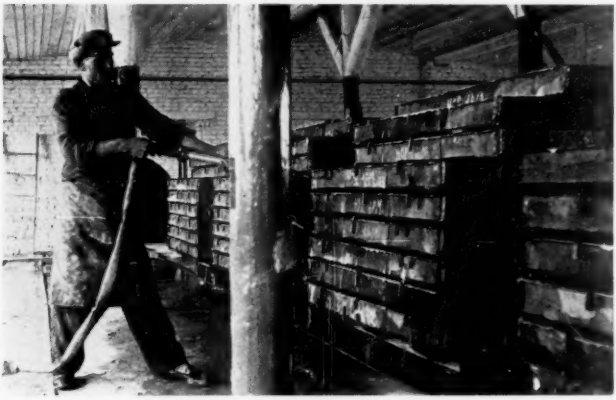

Fig. 3—The slabs were sprayed with water while curing, 8 to 10 hours after casting.

treated in the autoclave. Since the plant was working only one shift, the curing of the slabs actually continued for 16 to 24 hours. Eight or ten hours after casting, the slabs were sprayed with water from a hose (Fig. 3) using a special spraying nozzle to protect the soft surface of the slabs from the impact of the water jet.

When cured, the cars loaded with slabs were moved by a winch into the autoclave which was then closed and the steam turned on. The schedule used for steam treating was as follows: (a) raising steam pressure up to 8 atmospheres—3 hours; (b) curing under 8 atmospheres of pressure—6 hours; (c) lowering the steam pressure to zero—3 hours.

When steam treatment was completed, steam and condensed water were discharged, the autoclave opened, and in one or two hours the cars with the finished slabs were moved out.

Two hours after discharge from the autoclave the clamps at the corners of the forms were released, the forms overturned and removed from the slabs which were then transported to the storage yard.

Tests

To control unit weight and determine compressive strength, 10 cm (approx. 4 in.) cubes were made for each set of slabs when changing the brand of cement or the specified unit weight. Also, one slab was selected from each set to be tested for bending (Fig. 4 and 5). The physical characteristics of these slabs are given in Tables 1 and 2.

The results of bending tests for slabs with different unit weights and composition are given in Tables 3 and 4.

Fig. 4—One slab was selected from each set for bending tests.

Adopted mix

Reinforced foam concrete slabs with a unit weight of 750 kg per cu m (46.8 lb per cu ft) were adopted for mass production. The ratio of

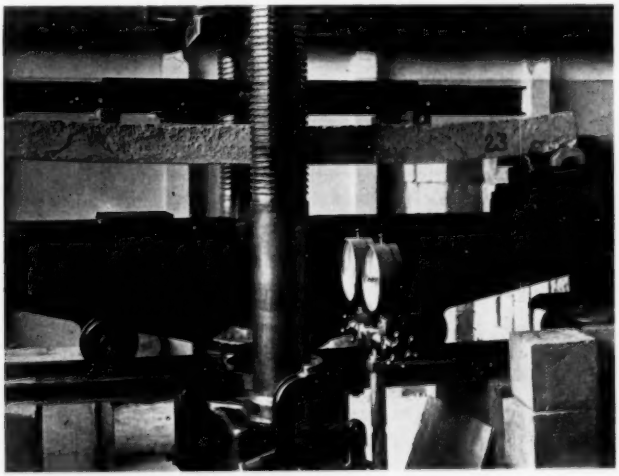

Fig. 5—Load test on autoclave treated slab

cement, marshalite and sand was 1:1:1 and the cement content 230 kg per cu m (4.1 sacks per cu yd).

PROPERTIES

The following investigations have been carried out with special reference to the effect of atmospheric conditions: (a) water absorption and

TABLE 1—PHYSICAL CHARACTERISTICS OF SERIES 1 TEST SLABS

No.	Materials per cu m (35.3 cu ft)						Unit weight		Size of slabs*			
	Cement		Marshalite		Sand				Width		Depth	
	kg	lb	kg	lb	kg	lb	kg per cu m	lb per cu ft	cm	in.	cm	in.
1†	220	484	220	484	—	—	500	31.2	50	19.7	8.5	3.4
2	270	594	150	330	150	330	630	39.4	50	19.7	9.5	3.8
3†	290	638	290	638	—	—	640	40.0	45	17.7	10	3.9
4	300	660	190	418	190	418	750	46.8	45	17.7	10	3.9
5	230	503	230	503	230	503	760	47.5	45	17.7	9.5	3.8
6†	235	517	235	517	235	517	780	48.7	50	19.7	10	3.9
7	195	429	195	429	195	429	650	40.6	50	19.7	10	3.9
8	155	341	223	491	223	491	660	41.2	50	19.7	10	3.9
9	155	341	155	341	310	684	680	42.5	50	19.7	10	3.9

*Length of each slab was 1.7 m (5.57 ft).
†Reinforcement in slabs 1, 2, 6, 7, 8 and 9 consists of 6 rods, 6 mm (¼ in.) diameter and 6 rods, 3 mm (⅛ in.) diameter. Slabs 3, 4 and 5 have 5 rods, 6 mm (¼ in.) diameter and 5 rods, 3 mm (⅛ in.) diameter.

TABLE 2—PHYSICAL CHARACTERISTICS OF SERIES 2 TEST SLABS

No.	Size of slab $l \times w \times d$		Weight of slab		Unit weight		Cement requirement	
	cm	in.	kg	lb	kg per cu m	lb per cu ft	kg per cu m	sacks per cu yd
1	166 x 49 x 12.5	65.4 x 19.3 x 4.9	69.5	153	550	34.4	195	3.5
2*	177 x 50 x 13	69.7 x 19.7 x 5.1	76.1	168	550	34.4	240	4.3
3	166 x 45 x 13	65.4 x 17.7 x 5.1	70.0	154	600	37.5	210	3.8
4	200 x 50 x 13	78.8 x 19.7 x 5.1	97.0	214	650	40.6	230	4.1
5	195 x 50 x 13	76.7 x 19.7 x 5.1	102.0	225	700	43.7	250	4.5
6	166 x 50 x 12.6	65.4 x 19.7 x 5.0	85.0	187	700	43.7	250	4.5
7	166 x 50 x 12.7	65.4 x 19.7 x 5.0	89.0	196	750	46.8	260	4.7
8	177 x 50 x 13	69.7 x 19.7 x 5.1	98.0	216	750	46.8	260	4.7
9	166 x 50 x 13	65.4 x 19.7 x 5.1	106.0	234	800	50.0	230	4.1
10	200 x 50 x 13	78.8 x 19.7 x 5.1	125.0	276	850	53.0	300	5.4
11	166 x 50 x 12.5	65.4 x 19.7 x 4.9	78.0	172	650	40.6	230	4.1

*The cement-sand ratio is 1:1.5 for all slabs except No. 2, which is 1:1.

frost resistance; (b) effects of alternate boiling water vapors and frost; (c) accelerated test for alternate wetting and drying; (d) determination of strength under saturated conditions (coefficient of softening); (e) determination of hygroscopic properties; (f) water condensation in the concrete and (g) the behavior of reinforced foam concrete slabs stored for 2½–3 years in the open air.

The strength and the coefficient of thermal conductivity, k, of the autoclave treated foam concrete, depending on the cement-sand ratio and on the properties of the components, are shown in Table 5 as measured in 10 cm (4 in.) cubes.

Full size test slabs, 2 x 0.5 x 0.12 m (79 x 19.75 x 4.75 in.) failed under uniformly distributed loads as shown in Table 6.

Compressive strength of the foam concrete increased 30 percent at the end of one year and 40 percent in 2½ years.

Investigations showed that the autoclave treated concrete was weatherproof (resistant to rain, snow, frost and heat) and kept its strength and volume constancy even under sharp temperature variations. Slabs

TABLE 3—TEST DATA FOR FOAM CONCRETE SLABS SERIES 1, LOAD UNIFORMLY DISTRIBUTED

No.	Compressive strength (10 cm cubes)		Loads§			
			At first crack		At failure	
	kg per sq cm	psi	kg per sq m	lb per sq ft	kg per sq m	lb per sq ft
1‡	17.5	249	—	—	1160	238
2	25.0	356	—	—	1400	287
3‡	32.0	455	—	—	2020	414
4	41.0	583	1230	252	*	*
5‡	42.0	598	1740	357	2230	457
6	43.0	612	1850	379	†	†
7	24.0	342	—	—	1300	267
8	24.0	342	—	—	1360	279
9	22.0	313	—	—	1180	242

*Slab No. 4 loaded to 1700 kg per sq m (348 lb per sq ft) without failure.
†Slab No. 6 loaded to 2350 kg per sq m (482 lb per sq ft) without failure.
‡Source and strength of cement used:
a. Slabs 1 and 2—Soucholojsky plant, compressive strength of standard mortar at 28 days — 240 kg per sq cm (3420 psi).
b. Slabs 3 and 4—Novorossijsk plant, compressive strength of standard mortar at 28 days — 280 kg per sq cm (3980 psi).
c. Other slabs—Katav-Ivanovsky plant, compressive strength of standard mortar at 28 days — 250 to 300 kg per sq cm (3560 to 4270 psi).
§Length of slabs is 170 cm (67 in.) while test span is 165 cm (65 in.).

TABLE 4—TEST DATA FOR FOAM CONCRETE SLABS, SERIES 2

No.	Age days	Load				Compressive strength (10 cm cubes)		Remarks
		At first crack		At failure				
		kg per sq m	lb per sq ft	kg per sq m	lb per sq ft	kg per sq cm	psi	
1	9	530	109	1000	205	20	284	
2	13	660	135	1180	242	20	284	
3	5	920	189	1540	316	27	384	
4	25	800	164	1200	246	30	427	Slab failed in 8 hours under 1200 kg per sq m (246 lb per sq ft)
5	44	....	...	2020	414	34	484	Cracks appeared at the moment of failure
6	23	1780	365	2050	420	..	...	
7	14	....	...	2475	508	38	540	A crack appeared at the moment of failure
8	18	....	...	2200	451	38	540	A crack appeared at the moment of failure
9	12	....	...	....	...	44	625	Slab did not fail under load of 2120 kg per sq m (435 lb per sq ft)
9	12	Impact load of 38 kg (84 lb) from a height of 75 cm (29.5 in.)				..	...	After the impact there were no cracks in the slab
9	12	The same load from a height of 100 cm (39.4 in.)				..	...	Shallow cracks appeared near the supports
9	12	2500	513	..	...	..	...	A crack appeared at the middle of the span in the zone of tension
10	3	1460	300	1735	356	52	740	Before test loading the slab was vibrated for 3 hours
11	9	Impact load of 38 kg (84 lb) from a height of 1 m (3.28 ft)				30	427	Deep cracks appeared in the zone of tension

TABLE 5—COMPRESSIVE STRENGTH AND THERMAL CONDUCTIVITY OF FOAM CONCRETE

Unit weight		Compressive strength at 2 days (10 cm cubes)		Thermal conductivity	
kg per cu m	lb per cu ft	kg per sq cm	psi	Cal per sq m per hr per deg C per m	BTU per sq ft per hr per deg F per in.
500	31.2	16 - 20	228 - 285	0.08 - 0.10	0.65 - 0.81
600	37.5	23 - 27	327 - 284	0.12 - 0.13	0.97 - 1.05
700	43.7	30 - 35	427 - 498	0.14 - 0.15	1.13 - 1.22
800	50.0	40 - 50	569 - 712	0.16 - 0.18	1.30 - 1.46
900	56.2	60 - 70	854 - 996	0.22 - 0.24	1.78 - 1.94

stored outdoors for 2½ years and exposed to all weather conditions showed neither cracks nor other evidences of failure nor a decrease in strength.

The coefficient of softening of the autoclave treated foam concrete, *i.e.*, the ratio between its strength in a water saturated condition and the normal strength, equaled 0.82, which is no lower than that of plain concrete and brick.

The autoclave treated foam concrete resisted 25 cycles of alternate saturation with water and freezing at –20 C (–4 F) without any decrease in strength or evidence of failure.

After 5 cycles of wetting for 48 hours, followed by drying at 70 C (158 F) for 48 hours, the concrete showed no evidence of failure or strength decrease. Owing to the treatment of foam concrete in an autoclave, the effects of shrinkage were comparatively small (not more than 0.5

TABLE 6—FAILURE OF FULL SIZE TEST SLABS*

Unit weight		Uniformly distributed load	
kg per cu m	lb per cu ft	kg per sq m	lb per sq ft
600	37.5	1400	288
700	43.7	1800	370
800	50.0	2200	452

*5.5 ft x 19.75 in. x 4.75 in.

mm per m, or 0.05 percent, at 1 to 2 years) and took place essentially while the concrete was hardening in the autoclave.

The water absorption of foam concrete with a unit weight of 600 kg per cu m (37.5 lb per cu ft) after soaking in water for 4 days, was 22 percent by volume or 36 percent by weight. The moisture content of the foam concrete was generally 4–6 percent.

When the autoclave treated concrete was subjected to a temperature of –30 C (–22 F) on one side of the slab and to a temperature of 18 C (65 F) on the other, accompanied by high humidity (up to 90 percent) the moisture condensation within the foam concrete at 1–1½ months was negligible (6–7 percent).

Exposure to fire at temperatures up to 350 C (662 F) for 4 hours failed to reduce its strength. Slabs 200 x 50 x 12 cm (79 x 19.75 x 4.75 in.) under direct action of fire for 4 hours, followed by wetting, exhibited a decrease in bending strength of only 35 percent.

The concrete developed a solid bond with the reinforcement, covering its whole surface with a watertight layer of very rich, dense mortar to resist corrosion.

CONCLUSION

Roofs built of these slabs and 2 layers of bitumen saturated felt, when substituted for the usual type of roof built of ribbed reinforced concrete slabs 4 to 5 cm (1½ to 2 in.) thick, an insulating layer of slag or foam concrete (not autoclave treated) and 2 layers of bitumen saturated felt, gave the following economies: (a) cost, 15 to 20 percent saving; (b) cement, 10 to 30 percent less; (c) metal, 20 to 40 percent less; (d) labor, 50 percent reduction and (e) time of completion was cut in half. The load carrying capacity and insulating properties of the two types of roof are considered equal.

As this issue was going to press the following comment arrived from the author—"After ten years' progress in the manufacture of cellular concrete the paper is considered unsuitable for publication." So far as we know, the method described has not been used in American practice, nor described in American technical literature. Despite later refinements in techniques it is believed to be of sufficient technical interest to warrant publication and at the same time avoid the necessity of re-makeup and delay in publication of this month's JOURNAL.

—Editor

Continuing the series of short articles on concrete floors, Mr. Gruenwald emphasizes that it is easy to build durable floors by observing the fundamentals of good construction practice and mix design.

Suggestions on Concrete Floor Construction*

By ERNST GRUENWALD†

SYNOPSIS

The relationship between good concrete floors and the proper selection of cement and aggregates is discussed. Data are cited to emphasize the advantage of coarse-aggregate mixes over cement-sand topping for concrete floors.

INTRODUCTION

Millions of square feet of concrete floors are in use today and have given excellent service for many years. Unfortunately, their fine performance record often is forgotten because of occasional poor results. But even these isolated cases need never have occurred, since it is easy to produce a good concrete floor.

Extensive literature exists on this subject, and much that will be said in this paper should be a familiar story. In 1930, as a result of an extensive study of concrete floors and their performance, ACI Committee 802 published a report[1] "Good Practice in Concrete Floor Finishing." The committee recommendations for concrete construction methods and proportions are still valid and by following them carefully an excellent floor can be produced. Both workmanship and construction procedure play an important part in attaining good results.

In building concrete floors, given good materials, workmanship is responsible for the final quality of the floor. The troweling must be timed carefully, since finishing wet mixes results in unsatisfactory surfaces. This phase of construction procedure is exhaustively treated in the above-mentioned report and elsewhere.‡

Since concrete floors are used under widely varying conditions, service requirements should determine the selection of the mix to produce the

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†Member American Concrete Institute, Manager, "Incor" Division and Technical Service, Lone Star Cement Corp., New York, N. Y.

‡See references 1, 2, 3, 5, 6, 7 and 8 at end of paper.

desired quality of concrete. The quality of the concrete depends on the quality of its ingredients, the proportions of cement to aggregates, the amount of mixing water, the finishing operations and, finally, curing.

MATERIALS

Cement

The properties of portland cements are well understood and compliance with A.S.T.M. Specifications is assurance of satisfactory quality. Types I, II, and III afford a choice for specific uses. Types I and II can generally be used interchangeably, with Type II offering somewhat increased resistance to sulfate waters. Type III—high early strength portland cement—offers the definite advantage of producing a floor ready for use after 24 hours and thus suggests itself for jobs where time is an important factor.

Aggregates

With aggregates also, compliance with A.S.T.M. Specifications assures generally satisfactory material. The aggregates should be selected for the service they are to give. A concrete floor must be resistant to abrasion, therefore the aggregates must have a high resistance to wear. The largest size and maximum amount of aggregate consistent with workability and thickness of floor topping should be used.

Table 1 shows guide values of fineness modulus for the total aggregate in relation to the cement content of the mix. These values have been prepared from a series of tests with good, average aggregate. One trial mix using the fineness modulus indicated for the particular cement content, with adjustments to allow for the characteristics of the aggregates used, usually will result in satisfactory proportions. While topping mixes containing coarse aggregates may not finish as easily as cement-sand mixes, the extra finishing work will be justified by the superior quality of the resulting concrete.

The size and shape of the aggregates have an important bearing on the amount of water required to produce a concrete of a given work-

TABLE 1—CEMENT CONTENT OF MIX AND FINENESS MODULUS OF TOTAL AGGREGATES

Size of aggregate	Cement content of mix—bags per cu yd					
	7	8	9	10	11	12
	Gravel as coarse aggregate*					
⅜-in.	4.58	4.68	4.78	4.88	4.97	5.06
½-in.	4.86	4.96	5.06	5.16	5.25	5.34

*For stone, reduce values by 0.1.

ability. For the same cement content, sand mixes require from 1 to 1½ gal. more water per bag of cement than a ⅜-in. gravel concrete, which results in a 1500 to 2500 psi reduction of the 28-day strength and a correspondingly lower resistance to wear.

Cement and water content

For a given aggregate, cement and water content of the mix determine quality of the concrete. Again, the service requirements control the selection of a suitable water content which should be no more than 5½ gal. per bag of cement for normal duty and not more than 4½ gal. per bag for heavy duty floors. Fig. 1 shows the familiar relation between the water content and compressive strength of concrete using "Incor" 24-hour cement and Lone Star portland cement. These strengths are from tests on 2-in. cubes rather than 6 x 12 in. cylinders, since it was thought that 2-in. cubes would indicate strengths more representative of floor toppings. The recommended water contents of 4½ and 5½ gal. per bag of Type I cement produced 28-day compressive strengths of 8000 and 5800 psi, as compared to 9000 and 7200 psi for Type III cement. Contractors specializing in concrete floor construction have used mixes with less than 4 gal. per bag testing as high as 14,000 psi at 28 days.*† Compressive strength may be taken as indicative of the

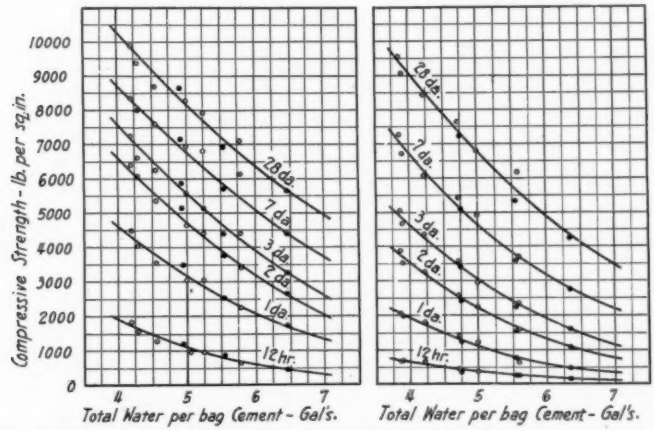

Fig. 1—Compressive strength of concrete floor topping mixes. "Incor" 24-hour cement (left) and Lone Star portland cement (right)

4-in. slump. Sand and ⅜-in. gravel as aggregates. Mixes by weight: 1-2, 1-2½, 1-3, 1-1-1, 1-1-1½, 1-1-2, 1-1½-2, 1-1½-2½ and 1-2-2½. Tests of 2 in. cubes, cured in water at 70 F. Strengths, average of 6 cubes.

*Test data, Cemasco Floor Co., Chicago, Ill.

†Mixes of cast stone are similar to the mixes used in floor toppings. The strength of 2-in. cubes has ranged as high as 21,720 psi. Tucker, J. Jr., Walker, G. W. and Swenson, J. A., "The Physical Properties of Cast Stone," ACI JOURNAL, Dec. 1931, *Proc.* V. 28, p. 253.

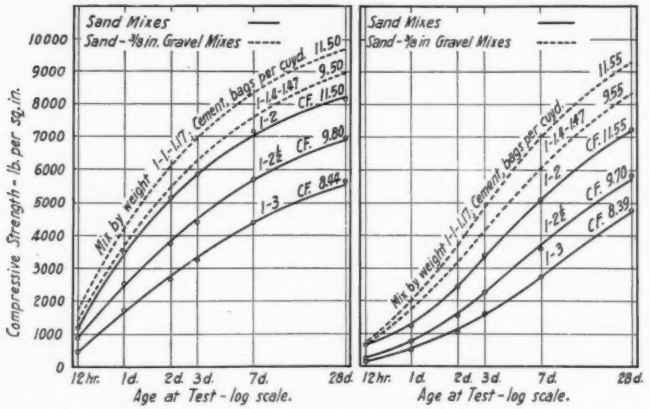
Sand Mixes
Sand-⅜in. Gravel Mixes
Compressive Strength - lb. per sq. in.
Mix by weight 1-1-1½ Cement, bags per cu yd 11.50
1-1½-1½ 9.50
1-2 CF. 11.50
1-2½ CF. 9.80
1-3 CF. 8.64
12 hr. 1d. 2d. 3d. 7d. 28d.
Age at Test - log scale.
Sand Mixes
Sand-⅜in Gravel Mixes
Mix by weight 1-1-1½ Cement, bags per cu yd 11.55
1-1½-1½ 9.55
1-2 CF. 11.55
1-2½ CF. 9.70
1-3 CF. 8.39
12 hr. 1d. 2d. 3d. 7d. 28d.
Age at Test - log scale.

Fig. 2—Compressive strength of cement-sand mixes. "Incor" 24-hour cement (left) and Lone Star portland cement (right)

4-in. slump. Fineness modulus of sand 2.60. Tests of 2 in. cubes, cured in water at 70 F, after 1 day in molds. Strengths, average of 6 cubes.

comparative wear resistance, as other investigators have found a general relationship between compressive strength and wear resistance.[4]

Proportioning

Because they are easy to place and finish, the old-fashioned 1:2 or 1:2½ cement-sand mixes are still widely used, despite technical data on strength, wear resistance, shrinkage and absorption showing them to be inferior to mixes containing coarse aggregate. In finishing a cement-sand mix the cement-water paste plus the fines in the sand are brought to the top, with crazing and dusting the unavoidable result. Furthermore, near the surface where the greatest wear-resistance is needed there is a layer of rich cement mortar of low resistance to abrasion.

Coarse aggregate, whether gravel or crushed stone, immeasurably improves a floor-topping concrete and, moreover, is more economical. Sand-gravel mixes containing almost two bags of cement less per cu yd of concrete will produce about the same strengths as the straight sand-mortar mixes, as shown in Fig. 2, using the previously mentioned cements. In addition, coarse-aggregate mixes have approximately 40 percent less shrinkage and show considerably less absorption than sand-cement mixes.

Proportioning of the mix must be done with great care. On bigger jobs, proportioning should be done by weighing the aggregates. Wheelbarrow scales may be used and on small jobs, where weight proportioning may not be undertaken economically, calibrated wheelbarrows or volume

boxes should be used. Lack of control in proportioning often is responsible for the varying quality of a concrete job, where areas of good concrete occasionally are interrupted by patches of poor quality.

CURING

Care in proportioning and good workmanship in placing and finishing are essential for a good concrete floor, but all this work may be nullified if curing of the freshly placed concrete is neglected. All concrete floors have a large surface for evaporation in relation to a relatively small mass of concrete. Rapid evaporation from the freshly placed concrete causes crazing and surface shrinkage cracks, vastly reducing the quality of the concrete. Concrete floor topping should be kept wet from 6 to 8 days when Type I or Type II cement is used, and from 1 to 2 days when Type III cement is used. Under average job conditions, where it is difficult to maintain continuous curing for 6 to 8 days, the use of high early strength cement is recommended. With its higher curing efficiency, one to two days of curing is sufficient to produce a durable floor. High early strength cement concrete cured one day has been shown to have the same wear resistance as Type I or II cement concrete cured 7 days. Tests have shown that high early strength cement mixes, after 1 day curing, have approximately 70 to 80 percent of their potential strength as compared to about 50 percent for Type I cement mixes.

SUMMARY

A well-built concrete floor will give long service over a wide range of conditions. The essential requirements for a good floor include appropriate construction methods and the use of adequate concrete mixes. Thus, building a durable, nondusting concrete floor resolves itself into the observance of a few fundamentals: select the proper cement, use suitable aggregates, proportion accurately, mix, place and finish carefully—and then cure thoroughly, because, other factors being equal, the better the curing the better the floor.

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Air entrainment in concrete continues to occupy the minds of engineers seeking to improve the quality of concrete and its placing and finishing. Mr. Waddell describes how air-entraining concrete was used advantageously in the lining of an irrigation canal.

Use of Air-Entraining Concrete in Canal Lining*

By JOSEPH J. WADDELL†

SYNOPSIS

This paper presents a discussion based on field observations of the use of air entrainment in canal lining concrete. The Bureau of Reclamation made studies of air-entraining agents for use in the irrigation canal lining for the Friant-Kern Canal on the Bureau's Central Valley Project in California. Results to date indicate that appreciable benefits accrue when an air-entraining agent is used in concrete which is placed and compacted by means of a mechanical slip-form. Care is necessary in adjusting concrete mixes to incorporate entrained air because of the sensitivity to mix changes of concrete for slip-form placement.

CONSTRUCTION PROCEDURE

New techniques, and the adaptation of old ones have characterized construction work on the Friant-Kern Canal, especially in the placing and subsequent treatment of the concrete lining. Concrete production and handling followed conventional methods of central plant batching, batch-truck hauling, and mixing in a dual drum paver. The machinery line-up for the canal construction is shown in Fig. 1. Continuous placing of concrete was accomplished by means of a self-propelled lining machine which shaped the concrete to the required contour. The machine was larger than any previously built at the time of its construction, being built to produce a bottom width of 36 ft, depth of 17 ft, and side slopes of 1¼ to 1. It traveled at a maximum speed of 110 ft per hour on 60-lb steel rails set accurately to a predetermined grade on the berms of the excavation. The trimming machine traveled on these same rails to accurately fine-grade the excavation. Most of the 75-ton weight of the lining machine, plus 25 tons of ballast, was supported by a slip-form of ½-in. steel plates which conformed to the canal cross section, pressing on the freshly vibrated concrete and shaping it. Operating power was obtained from two diesel engines mounted in the structural steel frame-

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†Member American Concrete Institute, Engineer, Bureau of Reclamation, Friant, Calif.

work, one of them driving the machine and the other driving an electric generator.

Maintaining grade

To maintain the form on grade, a steel wire was stretched on each berm, about two feet above the rail and a fixed distance above the canal grade. Four reference points, one over each truck of the liner, were held opposite the wires at all times by means of four hydraulic jacks built into the machine. By means of these jacks, a workman raised or lowered the form (Fig. 2) as the wheel trucks followed small irregularities in the track elevation. Thus it was possible to hold the finished grade within a tolerance of plus or minus ¼ in.

Lining characteristics

The lining is 3½ in. thick and except in the vicinity of certain structures it is unreinforced. Transverse grooves, 1 in. deep by about ¼ in. wide, were made at 12-ft intervals. Six longitudinal grooves of the same cross section were also made. These grooves were filled with a cold-applied mastic filler. Curing was accomplished by means of white-pigmented sealing compound sprayed on the concrete surface as soon as finishing had been completed and the grooves filled with mastic.

Placing concrete

Concrete from the mixer was carried in the boom bucket to an electrically driven shuttle car which traveled on rails across the form, from which it was discharged over a nearly vertical steel apron terminating in open bottom hoppers at the leading edge of the 4-ft steel skin plates comprising the form proper (Fig. 3 and 4). A subsequent improve-

Fig. 1—The line-up in the canal. From left to right are trimmer, sprinkling jumbo, lining machine, finishing and grooving jumbo, mastic joint filler jumbo, and curing compound jumbo.

Fig. 2—Operating one of the hydraulic jacks to maintain the form on grade

ment, made when the second machine was manufactured for this job, was the substitution of downspouts, or elephant trunks, for the apron. Nineteen 10-in. diameter spouts were suspended vertically in a row across the machine, their 48-in. square receiving hoppers forming a continuous receptacle into which the shuttle car discharged the concrete. Besides providing for better control of segregation of the concrete, the spouts afforded visibility through the machine up and down the canal. Six tube-type vibrators, situated perpendicularly to the canal axis and parallel to the bottom and sides, vibrated the concrete in place. These electrically driven vibrators were located immediately ahead of the skin-plate beneath the open bottom hoppers (Fig. 4). With 2¼-in. slump concrete, it was found that operation of the vibrators for 20 percent of the time was sufficient to compact it. Excessive vibration would cause the concrete on the slopes to slip towards the bottom, resulting in a wavy appearance of the finished surface. The concrete was not vibrated while it was under the slip-form, nor was the form vibrated.

Finishing

The concrete was semifinished as it emerged from under the slip-form, at times being rough and containing small voids or "bug holes" on the surface. A finishing operation was necessary to smooth the surface. Finishers, working on outriggers behind the form (Fig. 5), hand-finished the surface to a degree sufficient only to fill voids and produce a reason-

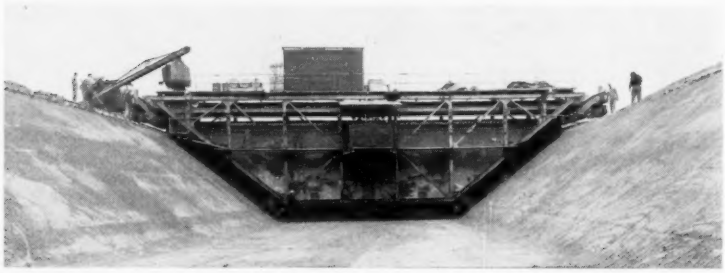


Fig. 3—Forward side of lining machine, showing the inclined apron and open bottom hoppers at the leading edge of the form

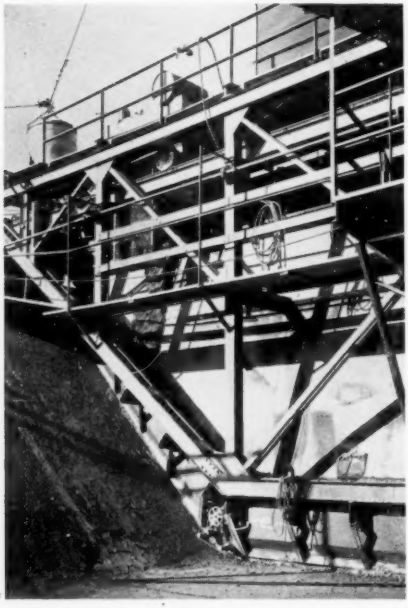


Fig. 4—Close-up view of the front of the liner. This shows concrete from the shuttle car being discharged over the steel apron and into the open hoppers. Motor drive for two of the vibrators may be seen at the lower edge of the liner.

ably smooth surface. Ordinary finishing trowels were used on the slopes, and long-handled steel floats were used for the invert. More work was necessary on the side slopes due principally to the fact that the concrete surface remained drier, any water resulting from bleeding finding its way into the invert. Only very slight working of the invert surfaces was necessary, one or two passes of the float being sufficient. Excessive finishing was harmful because of the presence of free water.

Fig. 5—Hand finishing the slopes on out-riggers behind the slip-form

AIR-ENTRAINING CONCRETE USED

The presence of this free water influenced the decision to try an air-entraining agent in the concrete. Because the canal is in a region where freezing and thawing are not significant, resistance to such weathering was not of prime importance. Other expected advantages of air entrainment, such as better workability and use of less water and sand, were considered to be of sufficient importance to warrant an attempt to make use of the relatively new procedure. The control of bleeding was the determining factor prompting a trial use of the material.

Finishing characteristics determine mix design

The first attempts to use an air-entraining agent were unsuccessful. This was due to the way in which the mix was adjusted; the procedure when adding an air-entraining agent was to reduce the amounts of sand, cement, and water in the mix, the theory being that concrete of equal workability could be obtained by reducing these constituents and at the same time entraining between 3 and 5 percent air. Results soon demonstrated that this was impractical so far as canal lining concrete is concerned. Equal workability was obtainable, it is true, but due to the peculiar finishing characteristics necessary for canal lining concrete,

it was soon found that "equal workability" at the time of placement was not the criterion by which to judge the quality of concrete.

Lining a canal with concrete using the method described requires that the concrete be finished immediately after placing. This finishing is extremely sensitive to the quality of the concrete. Slight variations in slump and workability affect the finishing characteristics much more than for concrete placed and finished by conventional methods. The first trials demonstrated the expected benefits of reduced water gain and reduced tendency towards segregation. Reduction of the amounts of sand, cement and water, however, counteracted any improvement that might have been noted in the finishing properties of the concrete after it had been deposited on the side slopes. The finishers complained of a lack of fines in the mix and stated that the concrete possessed a rubbery characteristic. That is, floating or troweling made a large area of the concrete quiver or tremble, causing it to slip down the slope. At times, cracks would appear in the fresh concrete. Deferring finishing until it could be done in a satisfactory manner delayed other operations, such as making weakened-plane grooves and application of sealing compounds, with subsequent increase in cost. Air-entraining concrete designed as mentioned above defeated the objective desired.

The writer recognizes that some of the effects attributed to air entrainment may have been due to other causes such as changes in aggregate grading, subgrade character, temperature, and others. Several trials with similar consequences, however, convinced all concerned that air entrainment was the chief cause of the effects noted.

Further trials proved successful

While finishing of the bottom flat was greatly expedited by the reduction in bleeding, there was a loss of workability on the slopes which more than offset the gain made in the invert. The next attempt was to add the agent to a concrete mix which was already workable, the only change being to adjust the amount of water to keep the slump constant. No adjustment was made to the amount of cement or sand. Such a mix adjustment was reflected in lower water and cement contents, the lower cement content, of course, resulting from the increased yield due to lessened unit weight of the concrete. Two such mixes are summarized in Table 1, with cement and aggregate properties listed in Tables 2 and 3. These particular mixes, containing 1½-in. maximum size aggregate, had a sand content equal to 40 percent of the total aggregate. (All mix proportions are by weight.) It was possible to reduce the sand to 38 percent of the aggregate, but any reduction below that point was sure to bring complaints from placing inspectors and finishers. This was demonstrated many times when the sand percentage was reduced without their knowledge.

TABLE 1—SUMMARY OF TRIAL MIXES

Percent air	Cement bbl per cu yd	Water lb per cu yd	W/C ratio	28-day compressive strength—psi
A: Mix proportions—1:2.71:4.06				
–	1.31	269	0.55	3350
4.3	1.25	259	0.55	3180
B: Mix proportions—1:2.53:3.79				
–	1.38	290	0.56	3470
3.3	1.34	276	0.55	3150

TABLE 2—PROPERTIES OF CEMENT
Type: Modified Type II

Chemical analysis		Compound composition	
Material	Percent	Material	Percent
SiO_2	24.6	C_3S	48.9
Al_2O_3	3.3	C_2S	33.9
Fe_2O_3	2.8	C_3A	4.1
CaO	65.4	C_4AF	8.5
MgO	1.4	$CaSO_4$	2.2
SO_3	1.3		
Ignition loss	0.61		
Na_2O	0.20		
K_2O	0.18		
Insoluble residue	0.11		

Physical properties	
Specific gravity	3.16
Autoclave expansion	0.04 percent
Compressive strength	7 days—2340 psi 28 days—3690 psi
Specific surface	1850 sq cm per gram

Air entrainment simplifies and speeds up finishing

At first glance, use of an air-entraining agent under these circumstances appears to be superfluous. Experience on the Friant-Kern Canal, however, was to the contrary. With a reduction of the unit weight of concrete from approximately 151 lb per cu ft to 144 lb, there is a 4.6 percent increase in yield per unit of cement, and it follows that there is a consequent reduction in cost per cu yd of concrete if the reduction in strength resulting from air entrainment is ignored. The increased workability and durability resulting from the entrained air made it possible to use a leaner mix having slightly less strength. Strength in this particular instance was ample and could be reduced accordingly. Finishing procedures were simplified and speeded up due to the reduction in water gain or bleeding and to improved workability. This was especially noticeable in the invert of the canal which was finished with

TABLE 3—PHYSICAL PROPERTIES OF AGGREGATE

Type: River-borne aggregate **Source: Madera Irrigation District Tract**

Item	Sand	Coarse aggregate		
		$\frac{3}{16}$ - $\frac{3}{8}$ in.	$\frac{3}{8}$ - $\frac{3}{4}$ in.	$\frac{3}{4}$ - 1$\frac{1}{2}$ in.
	Unit weight (lb per cu ft)			
Dry loose	96.7	92.6	91.4	93.6
Dry rodded	101.8	99.2	98.6	100.8
Mix proportions (percent)	40	5–9	19–23	32

Sand

Screen size	Percent retained
No. 8	11.1
No. 16	15.2
No. 30	31.2
No. 50	26.4
No. 100	12.0
Pan	4.1
F.M.	2.75

Color..... Clear

Silt..... 2.2 percent

Specific gravity..... 2.66

24-hour absorption..... 0.71 percent

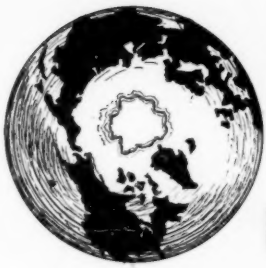
Sodium sulfate soundness ..... 7.1 percent (percent loss 5 cycles)

Coarse aggregate

Specific gravity.....2.65
24-hour absorption.....0.49 percent
Sodium sulfate soundness .....1.5 percent (percent loss 5 cycles)

long-handled steel trowels. Prior to the use of air-entraining concrete, bleeding of the concrete in the invert resulted in an excessive amount of free water which hindered finishing; at the same time, it was difficult to obtain a concrete that was rich and plastic enough to permit finishing the slopes without an excessive amount of hand labor. The old remedy had been to use oversanded mixes. Entrained air appeared to reconcile the conflicting requirements of low slump, high workability and minimum bleeding. The rubbery characteristic, noted in the first air-entraining mix, was eliminated in subsequent mixes. The air-entraining concrete was finished as soon as the slip-form had passed over it and seemed to cling to the slopes better than plain concrete. This latter was probably a function of the cohesiveness of the fresh concrete. Bleeding in the invert was virtually eliminated. Strength of the concrete was not affected to any serious extent.

Experience on the Friant-Kern Canal has demonstrated that concrete canal lining operations can be expedited by the judicious use of an air-entraining agent.



Letters from Readers*

BY WAY OF SYNOPSIS

SAM DAVIDSON, EDWARD E. BAUER and T. M. KELLY question a method of correcting for free moisture in aggregate presented in the May JOURNAL.

WILLIAM A. ANDREWS continues the discussion on the need for a redistribution of emphasis upon certain phases of teaching reinforced concrete design.

E. W. SCRIPTURE, JR. explains the difference between air entrained in ordinary concrete and that resulting from an air-entraining agent.

EIVIND HOGNESTAD presents some "behind the scenes" information on Scandanavian construction.

F. O. ANDEREGG suggests that discrepancies in air entrainment data on the effect of sand sizes may be reconciled when air bubble sizes are determined.

ROBERT C. STELZLE describes a method for determining the compressive strength of aggregates.

WALTER HESS asks for information on soil pressure.

J. NEIL MUSTARD describes dry-pack grouting as used by the Hydro-Electric Power Commission in Canada.

Making Corrections for Free Moisture (LR 45-19)†

Many readers were quick to call our attention to the error in the formula on p. 684 of the May 1949 JOURNAL. [Editor]

$$\frac{\text{Dry weight}}{(100.0 - \text{moisture})} = \frac{2000.00}{(100.0 - 0.08)} = 2174 \text{ lb}$$

should be corrected to read

$$\frac{\text{Dry weight}}{(100.0 - \text{percent moisture})} = \frac{2000.0}{(100.0 - 8.0)} = 2174 \text{ lb}$$

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 1, Sept. 1949, *Proceedings*, V. 46. Separate prints are available at 35 cents each. Address 7400 Second Boulevard, Detroit 2, Mich.

†See also ACI JOURNAL, May 1949, *Proc.* V. 45, p. 684.

SAM DAVIDSON, Denver, Colo., discusses free moisture correction further.

The method and analysis followed by Mr. Comack is correct, provided the percentage of free moisture is expressed as a percentage of the wet weight of the aggregate. If the moisture was expressed as a percentage of the saturated surface dry weight of the aggregate, which is the basis recommended by Committee 613 of the American Concrete Institute, then that which Mr. Comack says is incorrect is actually correct.

EDWARD E. BAUER, Univ. of Illinois, Urbana, Ill., emphasizes that in correcting for free moisture, values used should be consistent.

Mr. Henry Comack raises a question concerning the accuracy of the method generally used for computing corrections for free moisture in concrete aggregates.

It is standard American practice to base the percentage on dry weight of the material involved, not the dry weight plus the weight of free moisture. There is nothing fundamentally wrong with basing percentages on the combined weight of dry material and free moisture, but a person must know which method was used in securing the percentage and then use the value that way.

Mr. Comack's illustration begins with the value based on the dry weight only and ends up with the value based on the combined weight. When percentage values are small there is not much error due to an incorrect use, but as the percentage values become larger the errors become significant.

The 2000-lb weight used in Mr. Comack's illustration contains some moisture, if the 160-lb weight is considered to contain some aggregate. If it is desired to have 2000 lb of *dry* aggregate and the aggregate contains an amount of free water equal to 8 percent of the dry weight of the aggregate, then it is necessary to weigh out a total of 2160 lb, which will be composed of 2000 lb of dry material and 160 lb of water.

If the 8 percent value used in the illustration is based on the combined weight of water and dry aggregate, it is necessary to divide 2000 lb by 0.92 to secure the correct weight of wet aggregate to use which is 2174 lb.

T. M. KELLY, Beaverton, Ore., asks for a definition of terms.

Since mathematical inaccuracy is anathema to Mr. Comack, and since he is striving for mix designs that are "mathematically and analytically incontestable", he would do well to define more precisely his terms; such as indicating whether the free moisture was determined on

a wet or saturated surface dry basis. It would seem that the "dry weight" appearing in his last equation is in reality the "satfirated surface dry weight."

Teaching Reinforced Concrete Design—What Should We Emphasize? (LR 45-21)*

WILLIAM A. ANDREWS, Instructor in Civil Engineering, Washington Univ., St. Louis, Mo., makes some supplementary observations upon Prof. Holley's letter in the June JOURNAL.

This discussion can be limited to the undergraduate level for two reasons: (1) the great majority of men to be employed as reinforced concrete designers will not go beyond the Bachelor's degree and (2) graduate students will have sufficient interest to delve thoroughly and almost independently into the subject in its important phases. At the undergraduate level a definite interest must often be awakened in a student; and certainly any course based upon memorization of formulas will tend to be a burden on the typical undergraduate student. But aside from the mechanics of stimulating interest in the subject matter, it seems that Prof. Holley's suggestions have definite merit in fulfilling the needs of the design office.

The first prerequisite is an instructor familiar with design office practice from personal experience. Emphasis upon certain details will vary with the interest and experience of the instructor. Then certain specific techniques of analysis should be required of the student. Referring now to the straight line theory, the writer firmly believes that a student should be able to analyze any beam or column for internal stresses by cutting a section and applying the forces to the steel and concrete necessary to maintain equilibrium. In this manner the student learns how to solve the structure without recourse to a single formula. True, the student may never resort to such relatively long analyses in actual practice (only, twice has the writer found it necessary to rely upon such a procedure). But if this "rational" method of analysis is taught simultaneously with each type of structure the student analyzes by formulas, graphs, and transformed sections, certain advantages will result: (1) the student has a better understanding of the formulas and graphs he is using, (2) he becomes confident in his ability to study new ideas or theories and (3) when the student designs a building in the laboratory course the loose ends of the theory course are effectively gathered in. Considerable emphasis upon this rational means of analysis was introduced experimentally a few years ago at Washington University and has definitely resulted in a more effective course.

*See also ACI JOURNAL, June 1949, *Proc.* V. 45, p. 745.

It then becomes no major task to give brief instruction in some other theory than the straight line theory, and the idea is more effectively put across to the student's understanding that reinforced concrete analysis is not a static and fixed subject. In office practice in ordinary building design, possibly only four or five formulas are used day after day. Considering that these become second nature, and that other formulas are looked up, it may be said that memorization is a very small part of the designer's equipment. Therefore, is not the wisdom of memorizing formulas or code details subject to doubt?

There is a limitation upon the amount of time to be divided among the rational method, standard analysis procedure, and practical office procedure and correlation. The balance must be determined by each instructor experimentally. But the desirability of a more basic approach seems definitely proven.

What Is the Difference Between Air Entrained in Ordinary Concrete and That Resulting from an Air-Entraining Agent? (LR 45-22)*

E. W. SCRIPTURE, JR., Master Builders Co., Cleveland, Ohio, offers an answer to a question asked in the June JOURNAL.

While the work which we have done was not directly concerned with the questions raised by Mr. Anjel and does not provide definite data concerning them, it is possible to offer some tentative explanations.

Concrete made with a cement containing no air-entraining agent and with no air-entraining agent added at the mixer would be expected to contain a small amount of air, 0.5 to 1.5 percent, in the form of grossly entrapped air pockets as contrasted with entrained air in the form of "minute discrete spheroids" secured with air-entraining agents. Entrapped air in the form of relatively large bubbles probably does not have a beneficial effect on the concrete.

It is doubtful if there is any such thing as a cement completely free from air-entraining agent. Grease and oil leaking into the cement during grinding act, to a limited extent, as air-entraining agents. Current practice of making a run on air-entraining cement followed by a run on non-air-entraining cement undoubtedly causes the carry-over of air-entraining agent during the transition period which may persist to the end of the run on non-air-entraining cement. This is well illustrated by data on twelve brands of cement which varied from 4.0 to 7.6 percent air[1], when tested by A.S.T.M. Method C 185-47T. We have encountered non-air-entraining cements showing as high as 12 percent air in this

*See also ACI JOURNAL, June 1949, Proc. V. 45, p. 747.

test. The cement used in the work[2] referred to by Mr. Anjel showed 7.0 percent air in the A.S.T.M. mortar test. Probably the air contents in the concrete mixes using this cement with no agent added were made up of both grossly entrapped air and entrained air.

No determinations of resistance to freezing and thawing were made in connection with the work referred to but other data indicate that the resistance declines as the air content decreases below 3.5 to 3.0 percent. It would be expected that the mixes with no air-entraining agent added would in most, and probably all, cases show much lower resistance to freezing and thawing than the corresponding mixes with admixtures A, B and C.

From our work there is no information on the distribution of the total air content of the concrete between grossly entrapped air and entrained air, or whether the air-entraining agent reduces the size or quantity of the larger bubbles. Probably it does since the surface tension of the water is reduced by the agent tending to facilitate formation of smaller bubbles. It would seem that this could only be determined by measurement of the individual air bubbles in the hardened concrete by a method such as the camera lucida[3] method.

I believe, without direct experimental evidence, that in the mixes where air-entraining agents were added not more than 0.5 to 1.0 percent of the air was present in the form of grossly entrapped air. Some indirect evidence exists on this point in that good resistance to freezing and thawing has been found in mixes with an air-entraining agent containing 3.0 percent air in comparison with corresponding mixes with no addition having 1.5 percent air.

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1. Scripture, E. W., Jr. and Litwinowicz, F. J., "Effects of Mixing Time, Size of Batch, and Brand of Cement on Air Entrainment," ACI JOURNAL, May 1949, *Proc.* V. 45, p. 653.

2. Scripture, E. W., Jr. and Litwinowicz, F. J., "Some Factors Affecting Air Entrainment," ACI JOURNAL, Feb. 1949, *Proc.* V. 45, p. 433.

3. Verbeck, George J., "The Camera Lucida Method for Measuring Air Voids in Hardened Concrete," ACI JOURNAL, May 1947, *Proc.* V. 43, p. 1025.

Savings in Foreign Currency (LR 46-23)*

EIVIND HOGNESTAD, Univ. of Illinois, Urbana, Ill., adds some information to his review published in the May JOURNAL.

In my review of prestressed steel-tile structures in the May issue, the printed review ends, " . . . . represent a considerable saving as compared

*See ACI JOURNAL, Current Reviews, May 1949, *Proc.* V. 45, p. 689.

to other slab structures." Unfortunately, this is not so. I wrote: " . . . . represent a considerable saving *in foreign currency* as compared to . . . ."

The point is, Denmark, Sweden and Norway all have plenty of first class aggregates and raw materials for cement manufacture. All of them import much of their reinforcing steel. Norway and Sweden save all the wood, paper and cellulose they can to export as much as possible. Denmark imports wood. Thus, the building industry in all countries is advised to find ways and means to use as little *foreign* currency, in the form of steel or wood, as possible.

The steel-tile floors cost about the same as other slabs; but a far smaller portion of that cost is steel and wood—or foreign currency.

I agree that this is a strange way of doing things, but it seems to lead to recovery.

Bubble Sizes in Air-Entraining Concrete (LR 46-24)

F. O. ANDEREGG, John B. Pierce Foundation, Raritan, N. J., believes that the size distribution of air bubbles is important in correlating data on the effect of sand sizes on air entrainment.

Before the results obtained by different workers on the effect of sand sizes on the amount of air entrained in concrete can be adequately correlated, it is essential to have a good estimate of the size distribution of the bubbles of the entrained air. This size is controlled by several factors, such as reduction in surface tension and the presence of impalpable dust which goes to the interface and helps stiffen it. Then it is a simple geometric matter to show what size sand grain will best hold back a given bubble size. It is suggested that the apparent discrepancies reported in technical literature may well be reconciled when bubble sizes are determined and reported.

Determining Compressive Strength of Aggregates (LR 46-25)

ROBERT C. STELZLE, Cemenstone Corp., Pittsburgh, Pa., describes a method for determining the compressive strength of aggregates.

The ACI JOURNALS of April and May, 1949, carried articles* on lightweight aggregates. The method of determining the compressive strength

*Price, Walter H. and Cordon, William A., "Tests of Lightweight-Aggregate Concrete Designed for Monolithic Construction," ACI JOURNAL, Apr. 1949, *Proc.* V. 45, pp. 581-600.
Kluge, Ralph W., Sparks, Morris M. and Tuma, Edward C., "Lightweight-Aggregate Concrete," ACI JOURNAL, May 1949, *Proc.* V. 45, pp. 625-642.

of these aggregates is rather crude, and the results are indefinite. The method depended upon the compaction of the aggregate. The compaction varied from one to three inches, with a great variation in results.

It is true that by this method the weakest particles will be crushed first, and the progressively stronger particles will be crushed as the compaction increases. However, the tendency to crush is minimized by the restraint of the cylinder. If the compaction is carried far enough, the results obtained will depend upon the ability of the testing apparatus to withstand the forces of the compression. Furthermore, there is no point in the compaction which can be taken as the workable compressive strength of the aggregate.

A method used here to determine aggregate strength has been to make cylinders of varying strengths of cement paste and add the aggregate, always maintaining a workable mix. Then the cylinders are tested for compression. If the aggregate does not shear, the strength of the aggregate is greater than the compressive strength of the cylinder. When the aggregate does begin to shear, as the cylinder strengths increase, the compressive strength of the aggregate equals the compressive strength of the cylinder.

This method has the advantage that the results can be read directly in terms of the compressive strength of the concrete in which the aggregate is used. It is true that also in this method the weakest particles will shear first, but the technician may select by visual observation the point which he considers most representative of the average shearing strength of the aggregate.

Soil Pressure Under a Continuous Foundation Slab (LR 46-26)

WALTER HESS, East Chicago, Ind., presents a problem involving soil pressures with a continuous foundation slab.

In a problem involving a continuous foundation slab, reasonably flexible, under three or more column loads of varying intensities, are soil pressures a straight line diagram depending upon the eccentricity of the resultant load? Inverted, this becomes a continuous beam with soil pressures as loads, and column loads as reactions, and normally, loads produce certain reactions through the distribution of moments and shears. Do soil pressures vary to the extent of producing reactions equal to the column loads?

Dry-Pack Grouting (LR 46-27)

J. NEIL MUSTARD, Hydro-Electric Power Commission of Ontario, Canada, describes dry-pack grouting.

During the past year we have received several inquiries about dry-pack grouting and believe the brief description enclosed might be of interest.

The Hydro-Electric Power Commission has used dry-pack grouting for twenty years with very satisfactory results. "Dry-pack" means a 1:2½ mixture of cement and concrete sand, with just enough water added so that it will pack under pressure. The total amount of water added to the dry materials should not exceed 1¾ gal. per sack of cement.* Curing of dry-pack concrete is most important, so all exposed surfaces should be kept covered and moist for at least 10 days.

It is standard practice to use dry-pack concrete under billet plates and under heavy machinery which requires accurate alignment. The large bases of frequency changers and condensers are always shimmed to the exact position and then dry packed to distribute the load.

Dry-pack concrete is not affected by moisture and it is often used as a packing around the base of wooden power poles.

*One Canadian bag of cement weighs 87½ lb, one Imperial gal. weighs 10 lb.

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

Stresses in doubly reinforced beams

A. Feldman, *Concrete and Constructional Engineering*, V. 44, No. 3, Mar. 1949, pp. 81-84 Reviewed by Glenn Murphy

The feature of this article is a pair of charts designed to assist in determining the location of the neutral axis in a beam with or without compressive reinforcement. The charts are based on the standard theory.

Air entrainment

T. Heilmann, *Beton-Teknik* (Copenhagen), V. 15, No. 1, 1949, pp. 1-23 Reviewed by Eivind Hognestad

The article is based on a lecture given before the Danish Engineering Society. The author reviews the basic principles on which the functioning of air-entraining agents depends. The influences of entrained air on many physical properties of concrete are explained, and examples of practical applications are discussed.

A continuously-sliding mould

Concrete and Constructional Engineering, V. 44, No. 5, May 1949, pp. 159-160 Reviewed by Glenn Murphy

This article describes a sliding form used in Sweden for the erection of cylindrical concrete structures, such as silos. The top of the steel form consists of a platform to which the concrete is hoisted for distribution into the structure. No scaffolding is required and the form is raised hydraulically.

Reinforced concrete construction (Der Stahlbetonbau)

C. Kersten, Part I, 18th Edition, Wilhelm Ernst & Sohn, Berlin, 1948, 271 pp. 11 DM plus postage Reviewed by Rudolph Fischl

This textbook for school and office, well-known in Germany, was first published 42 years ago and has been brought up-to-date and enlarged in its 18th edition. A chapter dealing with precast reinforced concrete construction has been added. The easily understandable text gives numerous references for more detailed studies.

Simplified statics (Statik leicht verstaendlich dargestellt)

Zillich-Stiegler, Wilhelm Ernst & Sohn, Berlin, 1949,
Part I—Graphic Statics, 16th Edition, 74 pp. 3 DM plus postage
Part III—Larger Construction and Reinforced Concrete Construction, 15th Edition, 91 pp. 3.40 DM plus postage Reviewed by Rudolph Fischl

These two volumes are part of a five-volume textbook which treats statics in such a way that no basic knowledge is required to understand it. The text is in accordance with the curriculum of technical high schools in Germany.

*A part of copyrighted Journal of the American Concrete Institute, V. 21, No. 1, Sept. 1949, *Proceedings*, V. 46. Address 7400 Second Boulevard, Detroit 2, Mich.

Cementing agents and admixtures for mortar and concrete (Binde-und Zusatzmittel fuer Moertel und Beton)

B. WEDLER, Wilhelm Ernst & Sohn, Berlin, 1949, 47 pp. 1.80 DM plus postage

Reviewed by RUDOLPH FISCHL

This booklet is a collection of the German Industrial Standards dealing with cement, lime, gypsum and other agents and different admixtures.

Prestressed concrete (Hormigon elastico precomprimido)

JUAN GARCIA BALADO, *Hormigon Elastico* (Argentina), V. 1, No. 1, Nov. 1948

Reviewed by FRANCISCO J. CORDOVA

The author describes the development of the technique of prestressing concrete and enumerates its advantages. He is confident that prestressed concrete will be of great assistance to civil engineering and construction in Argentina.

Constitucion storage tank project (Un anteproyecto del deposito Constitucion)

RENATO LASTRA, *Hormigon Elastico* (Argentina) V. 1, No. 1, Nov. 1948, pp. 28-30

Reviewed by FRANCISCO J. CORDOVA

The article describes the engineering features of a 70,000 cu m prestressed concrete water storage tank designed for the city of Buenos Aires, Argentina. The interior diameter is 83 m and the maximum height is 43 m. A perspective drawing and two generalized plans are shown.

Reinforced concrete construction at new London stations

Concrete and Constructional Engineering, V. 44, No. 4, Apr. 1949, pp. 121-125

Reviewed by GLENN MURPHY

This is a progress report on the construction of stations on the westward expansion of the Central line of London Transport. One feature of the construction is a shallow reinforced concrete dome 44 ft in diameter. The 4-in. slab rests on 1½-in. wood-wool slabs used as permanent forms. Platform roofs are described in some detail and several excellent photographs are included.

Reconstruction of warehouse at Gladstone Dock, Liverpool

Concrete and Constructional Engineering, V. 44, No. 5, May 1949, pp. 149-154

Reivewed by GLENN MURPHY

The largest reinforced concrete warehouse at the Liverpool docks has been reconstructed following almost complete destruction during the war. The building consists of five sections each 300 ft long, 150 ft wide and three stories in height. The main beams have a 50-ft span and were cast in place, the reinforcing being assembled on the ground and hoisted into position as a unit. Secondary beams were precast. Construction details are given and several excellent photographs of erection are included.

Design of the ends of prestressed concrete beams

G. MAGNEL, *Concrete and Constructional Engineering*, V. 44, No. 5, May 1949, pp. 141-148

Reviewed by GLENN MURPHY

The author presents a method of computing the tensile stresses in the ends of a prestressed concrete beam and states that the method represents an improvement on the approximate method published in his recent book *Prestressed Concrete*.

The method is based on an assumed type of stress distribution and is illustrated for two examples for which direct comparisons with test results are available. The author states that the procedure is reasonable and safe, giving a factor of safety of two.

The rebuilding of Plougastel Bridge, France

Concrete and Constructional Engineering, V. 44, No. 4, Apr. 1949, pp. 119-120

Reviewed by GLENN MURPHY

At the time of its construction about 20 years ago, the Plougastel Bridge over the Elorn was the longest concrete arch bridge in the world, consisting of three arches each with a clear span of 590 ft 6 in. The structure was designed by Freyssinet and included a number of innovations. The north arch was demolished during the war and is now being reconstructed using essentially the same procedure that was employed in the original erection. Centering was floated into position on reinforced concrete pontoons.

A 1,600,000 gallons prestressed concrete elevated tank in France

Concrete and Constructional Engineering, V. 44, No. 3, Mar. 1949, pp. 95-100

Reviewed by GLENN MURPHY

The structure described is 148 ft long, 108 ft wide, 16½ ft deep, and is supported on 108 reinforced concrete columns. It is erected on the site of a demolished underground structure, the walls of which are used as footings for the 16-in. square external columns. Diagonal bracing is used between columns and a spherical hinge is used at the top of each column. The Freyssinet system of prestress used in the tank, and design details, as well as the construction features, are discussed.

Electricity and concrete

A. W. HASLETT, *New Zealand Engineering*, V. 4, No. 3, Mar. 10, 1949, p. 185

Reviewed by C. P. SIESS

The results of investigations concerning the effects of electrolysis on reinforced concrete are described briefly. Both the U. S. National Bureau of Standards tests and those made by the British Electrical and Allied Industries Research Assn. are discussed. It is concluded that leakage of AC "does no harm to concrete under any reasonable conditions" and that in the case of DC with the reinforcing bar as the cathode the bond strength may actually be increased.

Buildings with thin-slab vault roofs at Belfast

Concrete and Constructional Engineering, V. 44, No. 3, Mar. 1949, pp. 85-88

Reviewed by GLENN MURPHY

The tobacco factory buildings described consist of a bond store 420 ft long by 80 ft wide, a leaf-sample room 160 ft long by 90 ft wide, and a canteen 90 ft long by 60 ft wide. Reinforced concrete construction is used throughout and the roofs are thin-slab vaults, containing precast panels of lights. The structures rest on 17-in. concrete piles cast in hammer-driven steel tubes. The use of a standard 35-ft radius for the roof vaults made possible the repeated use of forms. Details of construction are given.

A non-destructive method of testing concrete during hardening

R. JONES, *Concrete and Constructional Engineering*, V. 44, No. 4, Apr. 1949, pp. 127-129

Reviewed by GLENN MURPHY

The method, developed at the Road Research Laboratory, consists in measuring the velocity of pressure waves through the concrete. Frequencies from 100 kc to 250 kc per sec were used, and it is stated that final setting of the concrete is accompanied by a rapid increase in velocity. An increase in compressive strength is associated with an increase in velocity. Curves are shown for two mixtures. No indication of repeatability of observations or spread of data is given.

Overbalancing in residual-liquidation computations

ALFRED S. NILES, National Advisory Committee for Aeronautics, *Technical Note* No. 1457, Feb. 1949, pp. 1-80

APPLIED MECHANICS REVIEWS May 1949 (Volterra)

This paper is an application of Southwell's relaxation method to statically indeterminate structures. By developing methods for the systematic selection and use of group displacements, the liquidation of residuals is expedited. Thus the amount of labor needed to obtain the desired result can be greatly reduced in many practical cases. The application of the method is illustrated by numerical examples.

The pressure distribution under bearing plates (Die Druckverteilung unter Lagerplatten)

G. HOFMANN, *Bauplanung und Bautechnik*, Nov. 1948, V. 2, pp. 313-314

APPLIED MECHANICS REVIEWS May 1949 (Durelli)

The author studies systematically by photoelastic means the distribution of stresses in semi-infinite plates compressed in their plane over a finite width of the free edge. The load is applied through blocks of varying heights and moduli of elasticity, and for each case the isochromatic lines are drawn. A table and diagrams give the best height-to-width ratio of the block to obtain the smallest stress concentration. The paper should be useful to foundation designers.

The effect of transverse grate reinforcement in compressed concrete elements (L'effet du frettage en nappes transversales des constructiones en beton arme)

WACLAW OLSZAK, *Preliminary Publication*, International Association of Bridge and Structural Engineering, 1948, pp. 247-266

APPLIED MECHANICS REVIEWS May 1949 (Durelli)

This paper deals with theoretical considerations on the behavior of concrete columns reinforced with transverse steel grates. Formulas and diagrams are presented which give the increase in longitudinal loading capacity of the columns as a function of the properties of the concrete and steel and the percentage of transverse reinforcement. Comparisons are made with some previous tests and with concrete specifications. The advantage of this method of construction in relation to the more conventional spirally reinforced concrete has not been made apparent.

Some simplifications in the numerical solution of Laplace's equation with special applications to photoelasticity

M. M. FROCHT, *Proceedings*, Society of Experimental Stress Analysis, V. 6, No. 1, 1948, pp. 39-43

APPLIED MECHANICS REVIEWS June 1949 (Wingren)

This paper deals with certain improvements in the numerical solution of Laplace's equation. A simple method, which the author calls the "Linear Rosette" method, is proposed. With this method initial values may be determined rapidly. When these values are used as key values in a square, functions may be determined along complete sections.

The paper refers to the author's *Photoelasticity*, V. II, and most of the information in the paper is included in Chapter 8 of this recently published book.

Concrete as a protective barrier for gamma rays from radium

HAROLD O. WYCKOFF and ROBERT J. KENNEDY, *Journal of Research*, National Bureau of Standards, V. 42, No. 5, May 1949

Reviewed by C. C. FISHBURN

Several papers and handbooks have been published during the past 20 years dealing with barrier requirements for shielding against gamma radiation from radium. These

publications usually have suggested lead for the barriers because a large proportion of the applications required restricted space and ready transportability of the barrier and gamma-ray source. For high-energy X-ray installations, it often is more economical to use concrete barriers. It has been suggested that concrete might also be useful for shielding in gamma-ray installations if attenuation curves were available. The present paper presents this required information for radium.

Arched beams in framed structures (Der gekrummte Balken als Teil von Rahmentragwerken)

GIACINTO TURRAZZA, *Bautechnik*, V. 25, No. 9, Sept. 1948, pp. 193-197 — APPLIED MECHANICS REVIEWS May 1949 (Newell)

This analysis of arched beams or girders in framed structures results in equations from which the end moments on columns and beams in single- or double-spaced frames may be computed. If tie rods are used between the ends of the arched members the loads in them may be obtained also.

The development may be followed readily by one familiar with the slope-deflection relations and the method of elastic loads, and the resulting equations are less cumbersome than those commonly encountered in the analysis of such frames. Terms which differ for different types of loading are given for usual types of loading. Solutions for two examples are included, carried to the point of setting up the algebraic expressions. A third example is solved numerically, the effect of a uniformly distributed load being considered first, and that of a twenty-degree rise in temperature second.

Pumice aggregate in New Mexico—Its uses and potentialities

DONN M. CLIPPINGER and WALTER E. GAY, *New Mexico Bureau of Mines Mineral Resources Bulletin* No. 28, 1947, 56 pp. $1.00 — CERAMIC ABSTRACTS June 1949

This bulletin describes and discusses the characteristics and uses of pumice, especially in construction. Data have been compiled from reliable sources concerning the successful uses of pumice aggregate and the physical properties of pumice products. Experiments also have been conducted using generally accepted particle distribution and concrete-mix designs. Graphs and charts comparing the physical properties of pumice with those of other building materials are presented. Special emphasis is placed on the importance of proper control in mining and grading operations to produce a satisfactory and marketable aggregate. Studies have been made of the economics of pumice concrete. The past and present production has been surveyed as an aid in predicting future demand. 25 figures, 21 references.

Investigation of formation of cracks in reinforced concrete structures

GEORG WASTLUND and PER OLAV JOHNSON, Swedish Cement and Concrete Research Institute (Stockholm), *Bulletin* No. 13, 1948

This bulletin adds new data to that reported in *Bulletin* No. 10 (See ACI JOURNAL, Sept. 1948, pp. 93-95), particularly the preliminary results of tests of T-beams under negative moments which showed that the width of crack is reduced when reinforcing bars are properly distributed across the whole flange of the beam. The results of tests of specimens subjected to positive moments are summarized as follows:

"Width of cracks increases almost linearly with the diameter of reinforcement bars; width of cracks becomes slightly smaller as the ratio of reinforcement increases; the use of deformed lug and helical bars, as compared with plain bars, reduced the width of cracks, assuming the diameters of reinforcement bars and the stresses set up in the reinforcement to be the same; and the compressive cube strength of concrete does not exert any notable influence on the width of cracks."

Determination of the permeability of hardened cements and concretes to water

L. N. KORZHUK, *Zavadskaya Lab.*, V. 14, No. 6, 1948, pp. 714-19

CERAMIC ABSTRACTS July 1949

The method was designed primarily for field tests; the results are expressed as coefficient of filtration. Metal tubes of 8-mm inside diameter are set in 7.07 x 7.07 x 7.07 or 20 x 20 x 20 cm cubes for at least 2 days and then kept in water for 26 days. The cubes are then kept in air for 2 days and dried to constant weight. The tubes are filled with water and connected to a high pressure system. A pressure of 10 to 15 atm is applied, and the time at which wet spots are formed on the surface of the cubes is recorded. Pressure is applied for 12 hours if wet spots are not observed. The system is then disconnected, the water is poured out of the tubes, the tubes are broken in half, and the diameter of the sphere of the water distribution is measured. If readings of three cubes differ by more than 10 percent, they are discarded. The coefficient of filtration, k, is calculated from $k = (a^3 - d^3)(a - d)/12adpt$, where a is the diameter of the sphere of the water distribution, d is the diameter of the tube, p is the pressure, and t is the time. Curves for making the calculations directly are also given. Sample calculations are made.

Evaluation of soundproofing of cement floor tiles (Schalltechnische Bewertung zementgebundener Fussbodenbelaege)

W. PIEPENBURG and H. BRANDT, *Betonstein Zeitung* (Wiesbaden), 1949, No. 4, pp. 60-63

Reviewed by RUDOLPH FISCHL

Based on numerous research works, basic recommendations for soundproofing in building construction have been published, but there still are unsolved problems. The physical conditions of transmitting and conducting sound are not at all explored. They do not follow elementary principles as do thermal conditions.

Tests with commercial cement floor tiles, built in two layers, the lower one of lightweight material with fair thermal properties, were made in the Laboratory for Testing Material at the Technical University in Braunschweig. The results are summarized as follows:

Soundproofing against transmission through air depends mainly upon the vibrations of the monolithic floor construction; that is, from the dead load per unit area.

Soundproofing against the noise of steps, moving furniture on the floor, etc. is highly dependent upon the configuration of the floor cover. Soundproofing can be improved by placing an intermediate elastic insulation.

The floor tile finish must be completely isolated from the structural floor and walls.

Insulation must be elastic with internal strength, with low absorption of moisture and should have a low thermal coefficient to serve also as a thermal insulation.

The Claerwen Dam

Concrete and Constructional Engineering, V. 44, No. 3, Mar. 1949, pp. 73-80

Reviewed by GLENN MURPHY

Construction has begun on what will be the largest concrete dam in Great Britain. The structure, across the valley of Afon Claerwen in central Wales, will form a lake four miles long with a storage capacity of 10,800 million gallons. The gravity section, which will contain about 400,000 cu yd of concrete is to be 1066 ft long, 224 ft high, and 180 ft wide at the base. The section is curved with a 2000-ft radius.

The structure is to be composed of 23 radial blocks, cast alternately, and separated by continuous copper strips. The lower part of the upstream face is to be brick lined and the remainder of the exposed surfaces covered with square rock-faced gritstone. The structure was designed to keep the thrust within the middle quarter of the base. Vertical pipes are embedded in the dam to relieve seepage pressure.

A 1:7 concrete using aggregate up to 6 in. is to be used in most of the structure. The coarse aggregate is brought from a quarry four miles away and a siliceous sand will be shipped to the site if necessary. A low-heat portland cement manufactured in Wales is used. Concrete from the 3-cu yd mixer is placed by means of a skip. The estimated cost is 6 million dollars. Details of the batching plant are described.

Prestressed concrete storage tanks in the United States (Despositos hechos en hormigon pretensado en E.E. U.U.)

M. R. Ros, *Hormigon Elastico* (Argentina), V. 1, No. 1, Nov. 1948 Reviewed by Francisco J. Cordova

In the United States the construction of prestressed concrete beams has not been as readily accepted as in Europe. Roy W. Carlson and K. P. Billner have developed a thermo-electric method of prestressing concrete. It consists of precoating the reinforcement with a sulphur mixture. Once the concrete is placed and set, the reinforcement is thermo-electrically elongated. However, by this method an insufficient stretching of the reinforcement is attained. Similar experiences by Ruml and others met with failure.

In the United States the greatest application of prestressed concrete has been in the construction of conduits and storage tanks. The method used for prestressing is similar to that of Vianini.

The prestressing method used by the Preload Corp. of New York is also discussed in this article. The major construction features of a storage tank built in Miami, Fla., are shown.

A comparative table shows the economics of prestressed concrete versus reinforced concrete.

Reinforcing of precast concrete joists with welded wire fabrics (BStg-Fertigbewehrung fuer Fertig-Betonbalken)

Ernst, *Betonstein Zeitung* (Wiesbaden), 1949, No. 3, pp. 35-41 Reviewed by Rudolph Fischl

Reinforcement with welded wire fabrics is applied to floor joists as described by the German Industrial Standards (DIN-Norm-Decke H1 and H4), but can be used for similar sections.

The standard floor system consists of concrete joists of reversed T-beam shape with concrete tile fillers. The longitudinal reinforcement in the joists is formed by one or two layers of welded wire fabric, the diameter of the longitudinal bars ranging from 4.2 mm to 12 mm with 4 bars at a spacing of 30 mm and transversal wire of 4.2 mm to 7 mm at a spacing of 300 mm. The web reinforcement is built of two wire fabrics with stirrups of 4.2 mm or 5 mm at spacings of 150 mm and 300 mm, respectively, connected by two 3 mm diameter longitudinal bars.

According to the German Specifications the allowable steel stress for this type of reinforcement is 28,440 psi.

Technical details for cutting and placing the wire fabric, design methods, tables for economical design at minimum weight of reinforcement and examples are given.

What kind of information does brittle coating give?

A. J. Durelli, *Production Engineering*, V. 19, June 1948 Applied Mechanics Reviews June 1949 (Boiten)

This article presents an analysis of the crack patterns of brittle coating ("Stresscoat") with the purpose of permitting a more accurate calculation of the stresses occurring. The information given by Stresscoat consists in the directions of principal stresses

(isostatics or stress trajectories) and in the isoentatics, or lines along which the principal strains are the same. The paper is based on the assumption that the elastic characteristic defining the failure of Stresscoat is the maximum tensile strain, although the author points out some exceptions to this law. By applying Mesnager's theorem and the more general Lamé-Maxwell equations several rules are given for determining the signs and the relative magnitudes of the stresses. Various zones of isostatic pattern may be distinguished. For each zone a method is presented for estimating the two principal strains ϵ_1 and ϵ_2 in plane stress, with the maximum value of the error.

The second part of the article deals with an example of the method, applied to the distribution of stresses in a ring under diametral compression. The results compare very satisfactorily with those obtained by photoelastic methods. A number of practical hints and suggestions may be useful to other workers with brittle lacquer methods.

The optical equations of three-dimensional photoelasticity

RAYMOND D. MINDLIN AND LAWRENCE E. GOODMAN, *Journal of Applied Physics*, V. 20, Jan. 1949, pp. 89-95

APPLIED MECHANICS REVIEWS May 1949 (Drucker)

In this paper Maxwell's electromagnetic equations are applied to two- and three-dimensional photoelasticity, and the results are compared with Neumann's equations. It is shown that the customary kinematic analysis of the propagation of light through two-dimensional models is exact only for homogeneous stress. The nature and magnitude of the terms which result from variation of stress in the plane of the model are investigated first, and the error in neglecting these terms is demonstrated to be negligible.

Ignoring them simplifies the subsequent study of wave propagation through a truly three-dimensional model. Here, as in the two-dimensional case, the order of the differential equations for the components of the magnetic vector is reduced by considering only forward traveling waves. However, it is now necessary to neglect a term involving the product of the rate of rotation of secondary principal axes along the wave normal and the relative retardation per unit length.

The two simultaneous first-order differential equations for the components of the magnetic vector along the secondary principal axes are much easier to handle than Neumann's equations. Reduction to canonical form shows that the ratio of the rate of rotation to the retardation per unit length along the wave normal in the absence of rotation is the controlling variable.

A new method of calculation for hydraulic structures of lightly reinforced concrete

P. P. LOUPMAN, *Civil Engineering* (London), V. 44, Jan. 1949, pp. 34-36

APPLIED MECHANICS REVIEWS May 1949 (Johnson)

This article is an abridged translation of a paper in the August 1947 issue of the Russian journal, *Gidrotekhnicheskoye Stroitelstvo*. The claim is made that more than half of all hydraulic structures are built of lightly reinforced concrete, because there is a weight saving in comparison with structures of mass concrete, and because there is an avoidance of hair cracks in comparison with heavily reinforced concrete under limiting tensile loads.

The existing standard specifications require that lightly reinforced concrete satisfy the same factors of safety for the formation of cracks and for ultimate strength as mass concrete; this leads, in effect, to a doubling of the factors of safety. The author proposes (plotting curves for illustration) that only at the mass concrete extreme should the safety factors with less and less reinforcing material match those for no reinforcement, and that only at the other extreme should the safety factors match those in the standard

specifications for regular reinforced concrete. The problem, of course, is to determine the intermediate safety factors.

The subject of lightly reinforced concrete will be fully described in the author's book now being prepared for printing; this article gives only fundamental data.

Elasticity beyond the elastic limit

M. Reiner, *American Journal of Mathematics*, Apr. 1948, V. 70, pp. 433-446

Applied Mechanics Reviews May 1949 (Allen)

The author begins by distinguishing between "deformation" and "strain" denoting by strain that part of a deformation which is recovered when all external forces are removed; however, his work applied equally to those cases when the so-defined strain and deformation are identical. On the assumption that there is a one-to-one relation between the stress tensor and the strain tensor it is shown that this relation can be expressed in forms

$$T^s{}_r = (F_{01} I_e + F_{02} II_e + F_{03} III_e)\,\delta^s{}_r + F_1\, e^s{}_r + F_2\, e \propto^s e_r \propto$$
$$e^s{}_r = (\Phi_{01} I_T + \Phi_{02} II_T + \Phi_{03} III_T)\,\delta^s{}_r + \Phi_1\, T^s{}_r + \Phi_2\, T \propto^s T_r \propto$$

where I_e, II_e, III_e, are invariants of the strain tensor, I_T, II_T, III_T, are invariants of the stress tensor, the F's are functions of the strain invariants and the Φ's are functions of the stress invariants.

The above relations are independent of any particular measure of strain, and the author chooses the Hencky (or logarithmic) measure in expanding his theory. By considering certain special types of stress field the F's are given meanings as generalized moduli of elasticity and the Φ's as generalized coefficients of elasticity. The reviewer finds some difficulty in appreciating the worth of these moduli and coefficients since neither F_{01}, F_{02} and F_{03} nor Φ_{01}, Φ_{02}, and Φ_{03} appear to be uniquely defined.

Concrete mixers and the homogenity of concrete (L'Homogeneite du beton et les betonnieres)

M. Albert Joisel, *Annales de L'Institut Technique du Batiment et des Travaux Publics* (Paris), No. 69, Mar.-Apr. 1949, 32 pp.

Author's Summary

Research on concrete mixers has not up till now been exploited as it should have been, as no theory has emerged which might establish principles for their use. Mixing is effected by the displacement of materials in a drum. It follows that the drums of mixers are circular and that mixing takes place during rotation. The displacement of materials is hindered by friction and particularly by viscous friction. Thus it is evident that mixing is facilitated by increasing the proportion of water, which decreases the viscosity. Gravity hinders such displacements as are perpendicular to it, and for this reason horizontal directions are advantageous.

The rotation of the drum or paddles causes the materials to move in planes which are perpendicular to the axis of rotation. This is why displacements in directions parallel to the axis are the most difficult. The criterion of efficiency of mixers is therefore the ease and speed of displacement of materials in directions parallel to the axis of rotation. Mixers with vertical axes (where the dimensions parallel to the axis are negligible) insure the greatest homogeneity. Concrete mixers with horizontal axes are more efficient than those with inclined axes.

Apart from materials and placing, the quality of concrete depends on the following in descending order of importance: proportion of cement, proportion of water, grading of aggregate. The constancy of the proportion of cement will therefore probably be the prime factor in the homogeneity of concrete.

Considerations on the principles of concrete proportioning (Nogle principielle betragtninger Vedr. fremgangsmaade og raekkefolge ved betonproportionering)

NIELS M. PLUM, *Bygningsingerioren* (Copenhagen), No. 4, 1948 Reviewed by DELMAR L. BLOEM and STANTON WALKER

The author's summary in English is too long to present in full and is not always easy to understand—a circumstance which would undoubtedly be improved if these reviewers could read the original Danish. Plum says that the proportioning of concrete involves the choice of definite values for: (1) cement content, (2) water content, (3) grading and shape of solids, (4) maximum size of aggregate, and (5) density and consistency of concrete.

He then summarizes (in the original) what appears to be the choice of various authorities as to sequence of determining these characteristics and says that it appears that the following order is preferred: (1) cement voids or void-cement ratio, (2) maximum size of aggregate, (3) density and/or consistency of concrete, (4) water content, (5) grading of solids (and their shape) and (6) cement content.

He proceeds to discuss these several factors and suggests that density, rather than consistency, be used as the criterion of workability. Based on discussions along these lines, he concludes that the preferred sequence of importance of factors affecting the proportioning of concrete are: (1) choice of compacting method, (2) maximum aggregate size, (3) concrete density, (4) estimation of the optimum combined grading, (5) estimation of consistency corresponding to choice of density, followed by several others. He follows with a brief discussion of each of these factors.

Acceleration of set and hardening of cements by heat (Acceleration de la prise et du durcissement des liants hydrauliques par la chaleur)

JACQUES BROCARD, *Annales de l'Institut Technique du Batiment et des Travaux Publics*, No. 54, New Series, Liants Hydrauliques, No. 4, Dec. 1948 AUTHOR'S SUMMARY

The main object of this investigation was to study the behavior of the principal cements on the French market during setting at temperatures above the normal to obviate mistakes in use on the part of contractors.

The first part of the investigation concerns the effect of temperature on the speed of setting and shows that the raising of temperature considerably shortens the setting time, both initially and finally.

Quick-setting cements, however, and particularly high-alumina cements and cements with high sulfate content were abnormal in behavior. Tests for hardening confirmed the conclusions drawn from the setting tests by showing that these cements had to be discounted for this type of work. They also showed that heating by electric resistance gave good results, but the cost of this method seemed too high.

Heating in water gave uneven results, especially for slag cements.

Heating in an autoclave gives interesting results, as much higher strengths can be obtained than in normal conditions of temperature and pressure. If the aggregate contains very fine siliceous elements, the strength is still further increased by a silico-calcareous effect.

The most convenient and economical method of hardening seems to be in saturated vapor at 80 C. The chief drawback of heating methods is the appearance of blisters on the cements. Luckily this anomaly can be avoided by preventing condensation forming on the surface of the test pieces.

Concrete in hydraulic structures (Betong i vannbygging)

INGE LYSE, *Betong* (Stockholm), V. 34, No. 1, 1949, pp. 1-14 Reviewed by EIVIND HOGNESTAD

This paper discusses the various modes of deterioration in concrete structures exposed to weathering. The main emphasis is given to damages caused by volume changes

and the disintegration due to freezing and thawing. The author believes that concrete exposed to sea as well as fresh water in Scandinavia is primarily deteriorated by repeated freezing and thawing. Possible chemical attacks generally are of secondary importance.

Results of recent researches on concrete durability are given, and the author summarizes his views regarding the principles on which a successful use of concrete in hydraulic structures should be based: concrete for exposed structures must consist of first class cement and aggregates, and must be so designed as to provide the necessary strength and impermeability. The workability of the concrete must be suitable for the method of placing to eliminate honeycombing and other irregularities. All reinforcement should be well covered with concrete. Air entrainment will increase the placeability and reduce the danger of separation. An air content of 3 to 6 percent is recommended.

Each concrete pouring should be limited to avoid excessive temperature rise. Construction joints should be treated to secure a good bond between the old and new concrete. The curing of the concrete is very important. Concrete poured in the spring and cured all summer will show considerably greater durability then the same concrete poured in late summer or fall. It is recommended that the concrete be allowed to harden for 12 hours before moisture for curing is provided. From then on moist curing should continue as long as possible.

On the strength of concrete pile tips

NILS ENGSTROM, *Betong* (Stockholm), V. 34, No. 1, 1949, pp. 30-49 Reviewed by EIVIND HOGNESTAD

Concrete piles usually are manufactured with tips consisting of a 120° pyramid covered with 2 mm steel plate. When piles are driven to rock, however, the piles often are equipped with a tip consisting of a steel rod welded to a plane plate 20 to 30 mm thick. When piles of these types are driven against rock or boulders, the pile tips often are damaged, while the pile itself remains sound. To investigate the strength of pile tips under these conditions, theoretical studies and laboratory tests were carried out.

The theoretical investigation shows that the failure under centric load is preceded by a splitting due to horizontal tensile stresses just above the pile point. Under eccentric loads, the failure occurs by local shearing.

The laboratory tests were performed on 25 by 25 cm square piles 1 m long. The specimens were made with tips at both ends. A total number of 60 specimens were tested involving 13 types of pile tips. In 21 of these tests, centric loads were applied while the remaining 39 tests were made with eccentric loads inclined 1:5 to the longitudinal axis.

The findings of the theoretical studies were confirmed by the tests. It was found that a considerable increase in strength of the pile tips under eccentric loads could be obtained if the pile end is made plane and furnished with a sheet steel pile shoe in the form of a box or mantle enclosing the tip.

Prestressed concrete (Hormigon elastico precomprimodo)

ZORISLAV FRANJETIC, *Hormigon Elastico* (Argentina), V. 1, No. 1, Nov. 1948, pp. 18-26 Reviewed by FRANCISCO J. CORDOVA

Franjetic describes several methods, including his own "Zofra" system, for prestressing concrete. The design of the prestressed units is based on the theory of equilibrium of the minimum stresses and on the laws of Hook and Bernoulli-Navier.

All prestressed concrete construction by the Zofra system uses steel wires 1 to 2 m in diameter prestressed to 50 percent of its breaking strength. Steel is saved inasmuch as the neutral axis of the prestressed reinforcement can be displaced by lowering it to a minimum, thus increasing the useful height of the member.

The making of a beam by this method is based on the principle of an arch under tension. This is attained by loading the concrete by compression and the steel by tension, with the total elimination of oblique stresses.

A beam is fabricated in two steps. First the lower part is made of prestressed concrete under tensional stress. In the second step, the upper part is made of plain concrete placed on the already set lower part. In the middle of the beam, between the parts under tensile and compressive stresses, a hollow span is made in the shape of an arch. Equilibrium is thus attained with the elimination of the upward bending of the beam under no load. In this form, the weight of the beam is reduced, increasing the bearing capacity and therefore a better use of the materials.

All prestressing operations by the Zofra system are conducted in an autoclave in which the free space is filled with water. In this way, the setting of the member takes place under water.

Symposium on hollow concrete masonry units

Cement och Betong (Malmö), V. 24, No. 1, 1949, pp. 10-50 Reviewed by EIVIND HOGNESTAD

Standardizing of hollow concrete masonry units

O. STUREN

Various viewpoints regarding the standardization of qualities and dimensions of hollow masonry units are presented. Several alternatives of length and width of units are discussed as well as special shapes like corner units, etc.

Measurements of the heat-insulation capacity of hollow concrete masonry units

P. JONELL

The article reviews an investigation carried out by Granholm regarding heat insulation of hollow masonry units. An "electrical analogy" is used in a study of the influence of the shape and dimensions of the cells or holes in the units. This analogy provides a simple method of comparing the heat transmission in various types of units.

The heat-insulation of hollow concrete masonry units

K. HELLBERG

The various factors influencing the heat transmission in a wall built up of hollow masonry units are discussed. A suggestion for a standard type of unit is presented.

Aggregate gradation for manufacture of hollow concrete masonry units

Suitable gradations for the manufacture of hollow concrete masonry units are discussed. Some advice and directions are given.

Structural details for walls of hollow concrete masonry units

Structural details are receiving increasing interest since hollow units are used more and more as outer walls in homes. Structural details from basement to roof are discussed.

Bond between concrete and metal (Liaison du beton et du metal)

L. P. BRICE, *Annales De L'Institut Technique Du Batiment Et Des Travaux Publics* (Paris), New Series, No. 72, May 1949, 20 pp. AUTHOR'S SUMMARY

This report describes investigations into the conditions of bond existing between metal reinforcement and concrete.

It deals first with the theory of the conditions for bond between concrete and the metal, and three hypothetical modes of behavior are envisaged: (1) an elastic bond, where deformations are reversible and in direct proportion to displacements, (2) frictional bond, where the stresses are constant in the opposite direction to displacements. The deformations are not reversible and (3) bond by means of successive anchorages, giving rise to a reversible phenomenon.

The second part is a report of experiments which were undertaken to determine the distribution of stresses along a reinforcing bar embedded in the concrete. For the experiments a steel tube was used, with strain gages with resistant wires fixed inside it at regular intervals. The gages measured the stresses in the steel in relation to the tension exerted across the gages. The result of the test shows that the nature of the phenomena causing the bond depends on the amount of displacement of the bar.

The elastic phenomenon applies for very small displacements and friction in larger displacements, and this seems to give way to bond by anchorages when alternating displacements have reduced the value of the frictional bond.

In the third part tests are described with a reinforced concrete beam in use under normal loads, and results confirmed that, in normal loading conditions, stressed concrete plays an important part in reducing tension in reinforcement. The position of the neutral axis in concrete is the same as that determined by ordinary calculations, but it appears that the deformations of the metal correspond to another position of the neutral axis which is close to the center of gravity of the total section, *i.e.*, without taking into account cracks in tension.

Effects of carbon dioxide on hydrated cement and concrete

S. L. Meyers, *Rock Products*, V. 52, No. 1, Jan. 1949, p. 96 Reviewed by T. M. Kelly

Considerable quantities of calcium hydroxide exist in cured concrete and concrete exposed to the air will slowly carbonate. Carbonation proceeds more rapidly in dry concrete, since the presence of water hinders the penetration of the carbon dioxide. Tests were made comparing the effect of carbon dioxide storage and carbon dioxide-free storage on the strength of specimens, and chemical tests were made to determine the extent of carbonation. The carbonated test pieces attained considerably higher strengths than these stored in carbon dioxide-free air. The percentage of CO_2 found in the center of these specimens (the least carbonated portion) was equivalent to more than twice the calcium hydroxide which cement can split off during complete hydration. The author concludes, therefore, that the carbon dioxide did break up one or more of the calcium compounds in the hydrated cement, other than the calcium hydroxide, and combined with the calcium of the compounds broken up. Additional carbonation tests were made on individual samples of the four principal cement compounds. Their rate of carbonation followed, somewhat, their rate of hydration. Probable reactions were presented showing the reaction products to be calcium carbonate and either hydrous silica, hydrous alumina or hydrous iron. The hydrous silica is soluble in a solution of sodium or potassium hydroxide and the possibility of its contributing to the expansive alkali-aggregate reaction is suggested. Tests of mortar briquettes showed that pore space was reduced by carbonation. It was also demonstrated by tests of mortar bars that the volume change of carbonated bars was only 30 to 40 percent of the volume change of uncarbonated bars, when both were subjected to the same condition of wet and dry storage. This was attributed to the replacement of some of the gel by calcium carbonate. The author suggests that concrete products manufacturers might utilize carbonation, after an initial hydration period, to reduce volume change, and to improve the strength, hardness and impermeability of their products.

A new approach to the study of concrete

NATHAN C. ROCKWOOD, *Rock Products*, V. 52, No. 5, May 1949, p. 60 Reviewed by T. M. KELLY

This is a thought-provoking article and, as the title indicates, proposes to cement and concrete researchers many new and to some extent heretical ideas. The clastic rocks produced by nature are suggested as a likely source of clues to the solution of concrete problems, particularly as regards cements. The author cites the preponderance of oxygen and silicon in the earth's lithosphere and the role of colloidal silica as a bonding material, inferring therefrom that a deficiency of reactive silica is perhaps one reason for the lack of permanency of present-day concrete. The evolution of hydraulic cements and the greater durability of some of the earlier cements are discussed, the latter being attributed to a better balance between the basic and acid compounds in the hydrated product. Evidence is referred to which indicates silica gel to be the most important end product of hardened hydrated cement. The inference then is drawn that for permanent concrete all voids should be kept filled with silica gel which ultimately attains a condition similar to chert, and which after a long time under certain conditions could become quartz. Based on this premise, a slowly hydrating, more coarsely ground, lower limed cement is considered best. It is suggested also that, since heat accelerates these phenomena, maximum heat of hydration would be desirable in the concrete in massive dams where a reaction silica or pozzolan is used. The shrinkage and cracking problem thus introduced is not discussed. It is suggested that the explanation of the alkali-aggregate problem may be found in soils chemistry, particularly from studies of base exchange phenomena. In regard to aggregates the author states that no aggregates are inert and attention should be directed toward making beneficial use of their reactions with cement. The interesting idea is proposed that, since alkali hydroxides are very effective in activating silica, a little of the hydroxides together with active silica, both well distributed in a portland cement, might make ideal cementing material.

Plain concrete footings

C. M. CROSIER, *Kansas Engineer*, Jan. 1949 Reviewed by FRANK KEREKES

Tests conducted by Crosier and McNown on small scale models of 22 wall and 22 column footings of plain concrete are described. Seventeen of the column footings were 24 in. square and the remaining 5 were 18 in. square at the base. The wall footings were 24 in. wide at the base and about 6 in. parallel to the wall. Both the wall and column footings were constructed in 3 equal steps with fillets at each set-back, except models A5 and A6. The footings were supported on about 2 in. of soft clay held in place by a wood frame of an appropriate depth to confine the soil. The author states that these footings were designed as true models and, therefore, their behavior under load should simulate the mechanical behavior of the practical sized prototypes.

These tests lead the author to conclude: (1) the critical stress is direct tension, (2) this critical stress may be calculated by the accepted flexure equation $f = \frac{Mc}{I}$ for cantilever beams, (3) the critical section of wall footings may be taken at the face of the wall, (4) the critical section for column footings may be taken at one-seventh of the width of the column inside the face of the column, and (5) a plain footing of high strength concrete with a riser-tread ratio of 1:1 will develop as high ultimate strength as a normally designed reinforced footing made of the same concrete.

Some practical design suggestions are: (1) constant ratio of riser to tread, (2) number of steps to use depends upon the economy desired, 3 steps are probably the practical maximum, (3) use high strength concrete for economy, (4) an allowable tensile stress

of 0.045 f'_c, less 0.005 f'_c for each increment of 1000 psi in f'_c of 3000 psi, and (5) the author does not recommend exceeding the current code specification of 0.03 f'_c, until further tests are made on larger footings, consisting of various aggregates and including sustained loadings.

The tests are well described and illustrated and the test data are carefully discussed and interpreted.

Gas concrete, foam concrete and lightweight lime concrete (Gasbeton, Schaumbeton, Leichtkalkbeton)

Otto Graf, Konrad Wittwer (Stuttgart), 1949, 76 pp. Reviewed by Rudolph Fischl

This booklet summarizes test results and experiences with three different kinds of lightweight concretes used as building materials in form of bricks and boards. The tests were started in 1940 at the Technical University of Stuttgart and partially reported in 1943 and 1944.

Three kinds of lightweight concrete have been tested:

1. Gas concrete, consisting of cement mortar with an admixture of any gas-producing matter which causes swelling of the concrete. The compressive strength is 30 to 40 kg per sq cm for a weight of 0.7 to 0.8 kg per cu dm.

2. Foam concrete where chemicals are used which produce foam during the mixing process, thus binding air bubbles to the mixture which loosen the concrete. Strength and weight are approximately the same as in gas concrete,

3. Lightweight lime concrete composed of ground silicic raw materials and ground lime, mixed with or without swelling compounds and cured under steam pressure. This concrete has much higher strength for the same weights than the gas concrete. The average compressive strength is 60 kg per sq cm for a weight of 0.7 kg per cu dm.

It is especially mentioned that the industrial production of lightweight lime concrete and gas concrete has its highest development in Sweden with a calculated capacity of 300,000 cu m each of lime concrete and gas concrete.

The reported tests cover a wide range, including influence of sizes of the test specimen, difference between weights and compressive strength, flexural strength, percentage of humidity, water absorption and expulsion, shrinkage, elasticity, thermal coefficient, sound insulation, resistance to freezing and thawing, hardening in air and under steam pressure, use as building material (bricks, boards) and many more.

Lightweight lime concrete rates first among the building materials suitable for bearing walls in buildings with several stories and for partitions, having a high insulation value.

The test results are carefully presented and supported by numerous graphs and tables, thus giving an excellent picture of the properties of the above mentioned materials.

Special cement to prevent cracking in hydraulic concrete structures (Specialcement mot sprickbildning i vattenbyggnads-betong)

Bertil Lofquist, *Betong* (Stockholm), V. 34, No. 1, 1949, pp. 15-25 Reviewed by Eivind Hognestad

An investigation carried out by the Swedish State Power Board indicates that the use of low-heat cement is particularly advantageous in structures of medium thickness, *i.e.*, with a thickness from one to a few meters.

The temperature rise is computed for various slabs ranging in thickness from 0 to 8 m by means of difference-equations on the basis of known heat evolution of the cements and assumed adiabatic boundary conditions. Normal portland as well as low-heat cements are considered.

Results of tests are then briefly presented, where specimens were subjected to the same temperature changes as those existing at the center of a 2-in. thick slab. Changes in length were measured under moisture conditions simulating those in the slab. The

maximum extension, which was obtained after about 4 days, was about 0.00046 and 0.00028 in. per in. for normal portland and low-heat cement respectively. A permanent extension of 0.00020 and 0.00014 in. per in., respectively, were present after the temperature returned to the original level.

A theory of the cause of through-temperature cracks is presented. If specimens as mentioned above are restrained while subjected to a temperature cycle, compression stresses will result during the expansion. The strains caused by such compression (strains equal and opposite the temperature expansion) are partly elastic, but mainly plastic. Thus tension will occur during the following drop in temperature. This theory, and the quantitative values involved are verified by test results. It is concluded that the tensile strain in the concrete after cooling will be approximately one half of the strain caused by the maximum temperature. Whenever this tensile strain exceeds the ultimate strain of the concrete, cracks will result.

Comparing normal and low-heat cement, it is found that the latter reduces the risk of cracking by at least 30 percent. The practical experiences of the State Power Board are in accord with this, even though a comparison between various structures generally is difficult.

The strength of concrete and its determination (La resistance du beton et sa mesure)

R. L'HERMITE, *Annales de L'Institut Technique du Batiment et des Travaux Publics* (Paris), New Series, No. 62, Concrete and Reinforced Concrete, No. 5, Feb. 1949

AUTHOR'S SUMMARY

The author examines the different tests which have been carried out on concrete to see to what extent the results obtained agree with the facts, or, in other words, what is the effect of the factors of placing, curing of test specimens, and test apparatus on the figures obtained. For this purpose he chiefly uses the statistical results of the compression tests and investigations undertaken in the Building and Public Works Laboratory.

Statistics show that, while cements today are on the whole inferior to pre-war cements (in the relation 0.9), they increase in strength more rapidly in the first few days (greater strength at 7 days); but it could not be said that there is a reduction in strength in the concrete between 7 and 28 days or between 28 and 90 days. Such reductions are due entirely to the wide dispersion of results, which is itself due to all the factors envisaged above, and probably they only appear to be more frequent than before the war because the strength curve of concrete is now more angular and more rapidly approaches the horizontal.

The author relies on chemical analysis to show that one cannot count on the amount of cement indicated, as cement is not distributed evenly in a beam. He recommends a simple apparatus for immediate analysis of fresh concrete, proposes a rational revision of traditional methods of curing cubes and draws attention to a crushing apparatus with self-aligning surfaces, which will soon be used for all test cubes in the Building Laboratories.

Lastly he describes a frequency apparatus for measuring the modulus of elasticity of concrete test specimens, which registers the modulus with great precision without the application of load.

This report is followed by a statistical investigation of tests on concrete carried out by the Bureau Securitas during 1947 and 1948. The investigation reveals a general reduction of breaking strength particularly during the second half of both years, greater sensitiveness of the mortar nowadays to excessive quantities of water and modified conditions of concrete hardening. However, the mediocrity of the results cannot be explained by the deficiency of cements, and in most cases such a deficiency can be compensated for by the care given to the placing of the concrete, as a typical example shows.